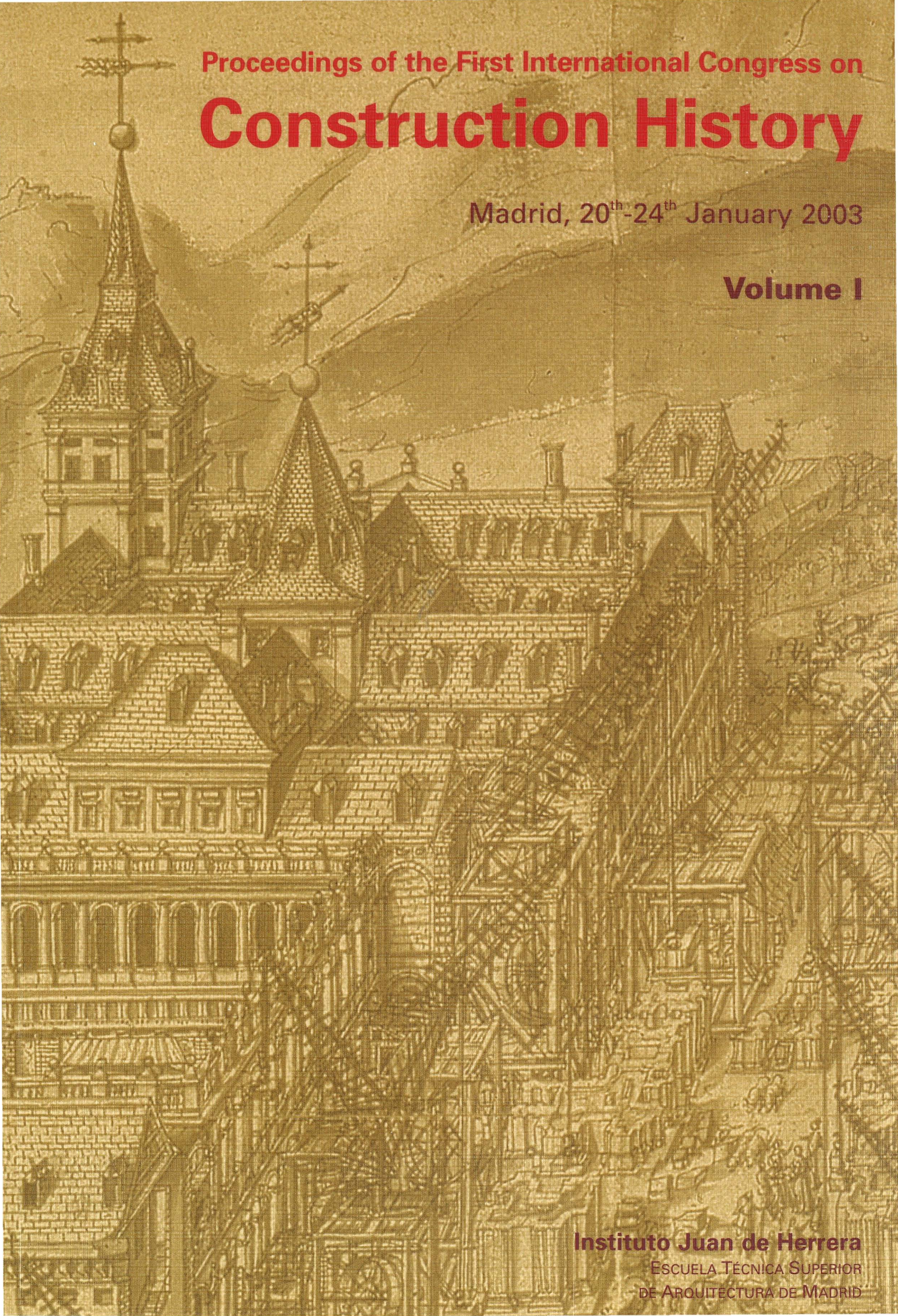
Proceedings of the First International Congress on

Construction History

Madrid, 20th-24th January 2003

Volume I



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First International Congress on

Construction History

FIRST INTERNATIONAL CONGRESS ON CONSTRUCTION HISTORY. Madrid, 20th – 24th January 2003

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Escuela Técnica Superior de Arquitectura de Madrid
Associazione Edoardo Benvenuto
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Proceedings of the First International Congress on

Construction History

Madrid, 20th-24th January 2003

Edited by
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Volume I

INSTITUTO JUAN DE HERRERA
Escuela Técnica Superior de Arquitectura
Madrid

ISBN 84–9728–070–9 (Obra completa)
ISBN 84–9728–071–7 (Volumen I)
Depósito Legal: M. 01.351–2003
The cover shows a fragment of a drawing on the building of the Escorial atributed to Fabricio Castello, 1576.
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Printing:
EFCA, S. A. Parque Industrial «Las Monjas»
28850 Torrejón de Ardoz (Madrid)

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Wren, Hooke and partners

Jacques Heyman

After the Great Fire of London, 1666, Christopher Wren was appointed Surveyor for the rebuilding of St Paul's Cathedral, and Robert Hooke was the Assistant Surveyor. Hooke was in his own right the Surveyor for the rebuilding of the destroyed City of London, and in practice the two men worked closely together on architectural projects —Hooke's diaries for the period record almost daily meetings with Wren, on site or in coffee-houses.

Building proceeded on an unprecedented scale, and medieval practice was forcibly abandoned. Instead of a «master», who made his own designs, employed his own men, and possibly had his own quarry for stone, the «gentleman architect» appeared as a Renaissance figure in England in the second half of the seventeenth century. Inigo Jones is the precursor, but Wren was the first real modern architect, a professor of astronomy who had never worked on a building site. Within a very few years Wren had established a recognizably modern architectural practice, with partners Hooke and Woodroffe, and later Hawksmoor and others, and together they were able not only to control the building of St Paul's (built at immense cost over 45 years), but also simultaneously to design and supervise construction of the 56 parish churches (51 in the City, 5 just outside), and other major buildings in and near London.

Further, in contrast to medieval design/build contracts with a single master, Wren and Partners provided the designs, and contractors were then employed in the modern way (most of these contractors were, initially, the medieval masters and men). Forms of contract had to be devised, and Wren used three types of contract still current today; the building work had to be assessed, and the modern quantity surveyor appears (the «measurer», as he was called in the Wren practice).

Thus the contractor —the successor to the medieval master builder— provided (from 1670 onwards) the technical expertise for building, including for example, the design of foundations. Visual appearance, however, was in the hands of the architect; and it was the architect who, on rare occasions only, explored the application of «science» —to the proper shape for a masonry dome, or to the required dimensions for the abutments of an arch bridge.

Medieval practice

The medieval way of building is illustrated by the work started in 1631 for St John's College, Oxford. Richard Maude, mason, entered into a contract with the College to design and construct a new range for the sum of £1.005. Some £700 had been spent when, in 1632, the scheme was enlarged; the range was now to form one side of a complete quadrangle (the Canterbury Quadrangle), and the new contract sum was increased to £3.200. Richard Maude possibly lacked resources, or he may have needed help with the design —he took as partners Robert Smith and Hugh Davies, and Hugh Davies is credited with the

design of the enlarged project. A year later, however, the partnership failed financially —the fixed-price contract had evidently been miscalculated by the masons. A new mason also failed within a few months, and in 1634 the College took charge of the work.

A Fellow of the College (John Lufton) now had direct control. He appointed a new master mason, John Jackson, and ran the operation with a mixture of direct labour, of payment for measured work, and of piece-work —arrangements to be used later in the century by Christopher Wren. Journeymen were scarce, and the College engaged the masters Timothy Strong and his son Valentine, paying them (secretly) at a higher rate than the local masons (the Strongs owned limestone quarries at Taynton and Little Barrington, neighbouring villages some 30 km from Oxford). Thus the Strongs, established master masons able to design and build using their own stone, were content to work as contractors (as the family was later to work for Wren on St Paul's Cathedral and the City churches).

Some thirty years later, Wren, as a young Savilian Professor of Astronomy at Oxford, turned his hand to architecture —first, in 1663, to the design (for his uncle, Bishop of Ely) of the chapel of Pembroke College, Cambridge, and then of the Sheldonian Theatre at Oxford (for which he contrived successfully the large-span timber trusses forming the roof). He then designed a new quadrangle for another Oxford College, Trinity, and first came into contact with the Strong family —the contractor was Timothy, grandson of Timothy and one of the six sons of Valentine, all of them masons.

The Great Fire, 1666

Gentleman (and scientists) other than Wren had diverted themselves with architecture earlier in the seventeenth century, but Wren must be reckoned to be the first modern architect; a person, that is, who was able and willing to take charge of the design of a major project without himself having served an apprenticeship in the trade. (Inigo Jones, an artist craftsman who came to fame as a stage designer, had a little earlier received commissions for classical architectural works, notably for the design of the Banqueting House, Whitehall, and for the great portico for Old St Paul's). Wren's learning came from books, and from a single year's stay in Paris, where he observed the buildings and monuments of that city, and, in particular, the domes, unknown in England. Lemercier had completed the 12 m dome of the Sorbonne in 1659. Mansard's 17 m dome for the church of Val de Grâce was finished by 1650, but his 28 m dome for the Invalides was not built until the end of the century. (Wren's dome for the new St Paul's was to span 34 m, a little more than that of Agia Sofia; Michelangelo's dome for St Peter's in Rome has a diameter of 42 m).

Wren had already been consulted about the state of Old St Paul's, which had been much ruined during the Commonwealth, and in Spring 1666, after his return from Paris, he proposed the construction of a central dome for the Cathedral. The Great Fire of 2 September 1666 put paid to this particular scheme, but not of course to the idea —in 1668 he was appointed architect for the new Cathedral. He had also been appointed immediately after the Fire as one of three Commissioners required to report on necessary rebuilding work in the City of London, and in 1670 he, with Robert Hooke and Edward Woodroffe, was made responsible for the rebuilding of the 56 City churches. At the same time Robert Hooke was the Surveyor responsible for the reconstruction of all other buildings in the City.

The scale of destruction, and of the consequent rebuilding, was enormous, and quite beyond the capacity of the existing skilled tradesmen in the City of London. An immediate measure was the Act of Parliament of 1666 which loosened the «closed shop» of the Masons' Company. Before the Act, only freemen, that is, citizens of London, could work as masons; from 1666 «foreigners» could come in from outside the City, and could work until the rebuilding was complete —in any case, they would be declared freemen after seven years of such employment. The Strong family took immediate advantage of this relaxation. Thomas, the eldest son of Valentine, and Edward, the fourth son, opened a shop near St Paul's Wharf, to which they shipped stone from their quarries in Taynton. The stone was for sale to any masons who needed it in the rebuilding programme, but the Strongs also offered their own services as contractor masons. The family prospered, and indeed Thomas was quickly made free of the Masons' Company, in 1670; he died in 1681, and thereafter

Edward, and his son also Edward, carried on the family business.

Wren's «partnership»

Despite the influx of working masons, there were too few «medieval» masters for the design of the 56 churches —the design of St Paul's, of course, came to be firmly in the hands of Wren, and this went through the well-known development and radical alteration before and after work started in 1675 with the laying (by Thomas Strong) of the foundation stone. The decision to demolish the ruins of Old St Paul's had been taken by 1668, but the work was laborious, and three years later Wren used gunpowder to bring down the mass masonry. Wren supervised four or five such explosions, but the last such attack was made in 1672, after Woodroffe had overestimated the required charge and caused damage to houses in the churchyard.

The incident gives a glimpse of the way the Wren partnership was developing. Woodroffe was officially Assistant Surveyor for St Paul's, but he was also, since 1662, Surveyor to the Dean and Chapter of Westminster; his work for St Paul's was charged to accounts for that cathedral. Similarly Hooke, in addition to his work for the partnership and his heavy duties for the Royal Society (not to mention his own substantial contributions to scientific research), also practised on his own account as an architect —major projects included the design and supervision of construction of Montagu House and of the great new Bethlehem Hospital in Moorfields. For these works he used staff in the Wren office —John Tillison, for example, who was the partnership manager responsible for payments and accounts, and for buying materials. (Woodroffe died in 1675 and Hooke was himself later to be Surveyor to the Dean and Chapter of Westminster, from 1691–97; Wren was appointed Surveyor for the repairs to Westminster Abbey from 1699).

Wren's arrangements with his partners were perhaps similar to those of today in a representative architects' office. That is, work for which the Wren office was clearly responsible was scrutinized by Wren —the 56 churches were in fact the responsibility of Wren, no matter which partner had actually made the design. Thus Wren's signature on drawings has until recently denied Hooke the design of some of the churches, but it is now clear that Hooke was the author of several (St Benet's, Paul's Wharf, built by Thomas and Edward Strong; St Edmund the King (steeple by Hawksmoor, a later «partner»); the tower of St Margaret Lothbury; probably St Martin Ludgate and St Peter upon Cornhill; and (demolished) St Michael Crooked Lane). Similarly, the Monument to the Great Fire, situated exactly at the north end of Old London Bridge (the bridge was moved slightly upstream in the nineteenth century), is usually attributed to Wren, but is actually the work of Hooke.

On the other hand the partners worked also on their own, and received personal fees (Hooke was paid £250 for Montagu House and £200 for Bedlam). Office facilities were used, and no doubt the office was reimbursed.

Contractual arrangements

Before the late seventeenth century in England there was no professional consultant architect between the client and builder. An Oxford college, as St John's in 1631, would seek a mason who was able to both design and build, and a design/build contract would be placed for an agreed fixed sum.

In fact, the consultant architect was not unknown in medieval times. William Hurley, King's Carpenter, designed in 1334 the unique and completely novel timber lantern for Ely Cathedral; the octagonal lantern was constructed by the local master carpenter, Geoffery de Middylton. Later in the century, in 1394, the King's Master Mason Henry Yeveley, was in charge of the remodelling of Westminster Hall (whose hammer beam roof was designed by another King's Carpenter, Hugh Herland). And a century later the designer and builder of the fan vaults of King's College Chapel in Cambridge, John Wastell, received advice from one at least of the Vertue brothers, who had built similar vaults at Bath and at Westminster Abbey. However, in all these instances the consultant architects were master builders —master was instructing master.

The anomalous arrangements for the Canterbury Quadrangle after 1634 were perhaps unique —the design had already been produced by the (now bankrupt) master builder, so that the client had no

need of a controlling architect, and could employ direct labour. However, once the designer was (in the modern sense) an architect, the client was compelled to employ builders willing to work under that architect, and the whole of the building trade was forced out of the medieval mould into something close to the modern form. That is, the architect would make drawings for the new work, and enter, on behalf of the client, into contractual arrangements with the builders.

This was, indeed, a major change in practice engendered by the emergence of the professional architect. The architect was in control of the construction of a building, but he no longer had personal financial responsibility, as did the medieval master builder —the medieval builder now worked under contract to the client, but was not finally responsible for the design. Contractual arrangements developed rapidly during the seventeenth century, and Wren's office evolved ways of handling large enterprises which are close to modern practice.

The medieval way of payment by the great (in grosso) survived in the form of fixed price contracts. The method was open to abuse —by a contractor asking for a large sum and making an excessive profit, or, alternatively, as at Canterbury Quadrangle, underestimating and failing financially. However, it was a method suitable for closely defined subcontracts —providing lead to cover a timber roof, say. From this had developed an unsatisfactory way of payment by valuation —the completed work was valued by an independent referee, and a high degree of trust was involved. Trust was also at a premium in payment by «daywork», although the method was used for tasks which were difficult to assess in advance (demolition, for example), but where the architect could exercise close control, and arrange payment in accordance with the number of man-days expended. Finally, payment by measure evolved as the most satisfactory way of proceeding with large contracts, a method very close to that used today —unit rates were fixed with the builder for the separate items in the contract, and the finished work was then assessed, both by quality and by quantity, by the «measurer». The quantity surveyor is nowadays an independent professional, but Wren employed in his office his own measurer, John Scarborough, and Scarborough was used by the partners in their «private» work (e.g. by Hooke during the construction of Bedlam). Scarborough indeed made surveys on his own account on other building projects.

With this apparatus in place, Wren was able to deploy large numbers of men on the site of St Paul's —at times there were two or three main master contractors (for example the King's Master Mason, Joshua Marshall, and Thomas Strong were both engaged from the start of the work, and up to two-hundred men, journeymen and labourers, were employed). The wage bill was high, and cash flow was more of less of a problem during the whole building period; after twenty years the St Paul's Building Fund was deeply in debt. The debts were in the main to the principal contractors, and in 1694 these contractors made an extraordinary proposal —that the debts should be considered as loans from themselves to the Building Fund. Work could then proceed with the loans attracting interest at six per cent, and over £24.000 was transferred in the books from debt to loan. Strong, for example, was recorded as being owed the huge sum of £2.500 —the contractors had become rich men. But not disproportionately rich; £24.000 on an expenditure (by 1694) of say £1/2 M is about five per cent, and the contractors' profits were perfectly reasonable. Thus work continued, and a new Act of 1697 gave some measure of future financial security; indeed, the contractors were eventually repaid.

THE INPUT OF SCIENCE

Inigo Jones was an artist who as a young man had spent several years in Italy; as a result, he had mastered Italian styles (above all, that of Palladio), and from the age of about thirty he became noted as a theatrical designer, before eventually moving on to the design of real buildings instead of painted scenery. Scarborough, the measurer, seems to have been a practical clerk of works. Little is known of Woodroffe, who died in 1675; his successor, Hawksmoor, joined the practice at the age of eighteen in about 1679 as Wren's clerk. In effect, Hawksmoor is the first English architect to have been trained in an architectural practice —perforce, since no architectural practice had existed before that of Wren.

By contrast, Wren and Hooke were gentlemen (Wren more gentle than Hooke), and, by training, scientists. Both were regular weekly attendees at

meetings of the new Royal Society, and in fact the Royal Society only survived by the skin of its teeth through the appointment in 1677 of Hooke as its permanent Secretary. Wren, as senior partner of his practice, was faced with the whole range of administrative problems familiar to the present-day architect, from the supervision of the work of his clerks and consultations with his partners, to the choice of materials (involving visits to quarries), and to the assurance of continued finance —at the same time he made contributions to science. Hooke, as Curator of Experiments and later Secretary was the one paid professional scientist of the Royal Society; his duties were heavy, and at the same time he carried on his own scientific research and his own contributions to architecture (including his almost continuous survey work for the rebuilding of the City).

With one or two exceptions, however, there are few examples of any scientific input from Wren and Hooke to their architectural practice. One overwhelming reason for this lies in the properties of the materials they were using, namely stone and wood. A stone structure, from a small arch bridge to a vast cathedral, does not experience high stress —that is, stone structures do not rely for their safety on the strength of the material. It would be a century before iron was used in a real sense as a structural material (Coalbrookdale) and another century again before steel and then reinforced concrete were designed as skeletons for structural forms. These «modern» materials are used sparingly, and are worked much harder than masonry, with stresses rising to a considerable fraction of their limits. Thus poor quality stone (as far as strength is concerned) can be perfectly satisfactory for a large masonry building, and modern ideas of strength are not of first importance. Ancient and medieval rules for the design of masonry —from Greek temples through Vitruvius to Gothic cathedrals— were concerned with geometry; it is a correct shape rather than strength that confers stability.

This concentration on geometry was fundamental to architectural thought, and the explicit rules of Vitruvius carried through to the Renaissance of Roman architecture; ideas of forces within a structure were not considered in the seventeenth century (Galileo had of course discussed in 1638 the breaking of a cantilevered beam). Wren knew that arches require support from their abutments, but his analysis of the problem is based on shapes and proportions, and he does not come to grips with the idea of force as a vector. Indeed, there is no real need for a «modern» analysis until the arch span becomes very large; small arches, Palladian villas, and classical porticos for cathedrals all have a very large «factor of safety» as regards shape. That is, a building designed in accordance with classical rules of geometry could in fact distort those rules to satisfy the whim of the architect, and still remain a perfectly viable structure. Thus Inigo Jones, having mastered classical proportions in order to design scenery for the stage, could transfer those designs with impunity to the real world.

When spans are large, however, whether of the two-dimensional arch or the three-dimensional dome, then the latitude of the architect becomes restricted, and it is of importance to try to understand more clearly the genesis of the rules of proportion. Hooke's famous solution of the problem of the arch is one of determining a correct geometry —his Latin anagram published in 1675 is concerned with «the true Mathematical and Mechanical force of all manner of Arches for Building, with the true butment necessary to each of them» (the last clause claims the solution to the problem of the abutment thrust). The anagram, deciphered and translated gives «as hangs the flexible line, so but inverted will stand a rigid arch». In other words, a light string loaded with given weights, will take up a certain shape —that same shape, inverted, is that of the perfect arch to carry the same weights. The shape, for uniform loading, is that of the catenary, but Hooke and his contemporaries at the Royal Society were unable at that time to obtain the shape mathematically.

Hooke had stated his principle to the Royal Society in 1670 and, a year later he extended his analysis from the two-dimensional arch to the dome (of uniform thickness). He stated, without proof, that the perfect shape was that of a cubico-parabolical conoid (the curve $y = ax^3$ rotated about the y-axis). (The statement is incorrect, but the difference between Hooke's surface and the exact mathematical surface is very small indeed.)

Wren had experimented with a small dome (13 m span) for St Stephen Walbrook (1672–9; mason Thomas Strong). This dome, however, posed no structural problems —it was built from laths and plaster, and was more like a design executed as a

stage set. As is well known, Wren made several radically different designs for his 34 m dome for St Paul's, refining his ideas over a period of thirty years. His own 1693 sketch for a triple masonry dome (not built) shows the middle of the three domes with the profile $y = ax^3$, explicitly related to marked axes. The essential point of Hooke's principle is that the hanging chain, and the three-dimensional equivalent inverted dome, can never become vertical at the abutments; the drum supporting a dome should slope inwards rather than be vertical. Wren's final dome for St Paul's has a timber outer structure; the two inner domes are of masonry, and remain sloping as they approach the supporting drum.

Just as the design of medieval stone buildings was in the hands of master masons, so the design of timber roofs was made by master carpenters. Medieval building manuals —for example, that of Villard de Honnecourt— thought it was worthwhile to record the design of roofs; the «secrets» were those of proper triangulation and of arranging for individual members to be, as far as possible, in compression. These principles are sound, and were not too difficult to achieve in a reasonably small church, whose massive masonry walls could resist the thrust from an arched timber truss. The design for the Sheldonian Theatre, however, raised problems because of the large span, over 20 m. Wren adopted a flat triangulated truss form, imposing no lateral thrust on the supporting walls, but he had to devise effective tension members 20 m long from much shorter lengths of timber. He used ingenious scarfed joints, very reminiscent of those shown in the first illustrated edition of Alberti, published (in Italian) in 1550. (The first English edition of the Ten books of architecture was not published until 1726, but Wren owned a Latin edition of 1512.)

Another borrowing from Alberti concerns the use of inverted arches buried underground to spread the loads more evenly to continuous foundations. Hooke used Alberti's idea in 1675 for his design of Montagu House, and Wren did the same in his Library for Trinity College, Cambridge.

Envoi

It is unrewarding to assign authorship of some of the buildings designed by Wren, Hooke, Woodroffe, Hawksmoor and all the other clerks in their office, not to mention the contributions that must have been made by the experienced master masons they employed, and by the artists engaged to embellish their work. It is clear that Hooke was responsible for the idea of the proper shapes for arches and domes, and there is no record of any dispute about this; and there are clear instances where the hand of one partner rather than another had made the major contribution to a particular design.

One of the advantages of a partnership is that problems can be discussed freely in the office —when a solution has been found, it may be difficult to attribute that solution to any one person. Thus, at the weekly meetings of the newly formed Royal Society, the question of universal gravitation was a frequent subject of informed speculation, and the inverse square law was discussed by many members over several years. Indeed, Hooke had published the law as being certainly correct but, as with his cubico-parabolical conoid, without proof: he was nevertheless outraged when Newton's Principia appeared a few years later without acknowledgement to him. The fact is that Newton had proved, but for himself only and before Hooke's publication, that the inverse square law explained planetary motion —and, more importantly, Newton had been able to do the mathematics, a feat totally beyond Hooke and the other members of the Royal Society. Newton had found some smooth pebbles and pretty shells (to use his own metaphor) and assembled them into a giant intellectual structure —it was, for him, irrelevant whether or not others had handled those pebbles before.

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Invited lectures

Establishing a postgraduate programme in Construction History. The experience of South Bank University, London

Malcolm Dunkeld

Building history in British universities has been subject to a number of pedagogic approaches. In schools of architecture it is generally taught in terms of the stylistic treatment of buildings, being about great buildings designed by important architects. The topics covered typically consist of: Greece and Rome, Medieval building, Florence in the 15th century, Mannerism and Baroque architecture, French and English 17th century buildings, neo-classicism and 18th urbanisation, the Industrial Revolution and finally the problems of architecture, planning and building at the start of the 19th century.

This approach elevates the stylistic treatment of buildings above all other considerations (and in so doing reinforces the importance of the architect) and is often taught in the classroom using a succession of colour slides detailing the peculiarities of different styles. Academically such an approach is justified on two grounds: it introduces the student to the realm of culture, and it provides rich visual material for the student to incorporate into their designs. The student becomes acquainted with a standardised history generally accepted within the architectural profession as a literary «rite de passage».

The teaching of architectural history tends to answer the questions «what happened?» and «how?», but pointedly avoids the question «why?». As a result, the change from one style to another is discussed in terms of the «exhaustion» of that style, or in terms of personal relations —architects «reacted against» their fathers. What is missing from this approach to building history is any recognition of the industrial setting that allowed earlier societies to sometimes build on a large scale. While it is true that architecture is related to its social function, it is also connected to the productive forces that brought it into existence.

Another approach to building history found in the universities is what Hamilton[1] has called the «cultural field . . . (of) one's own profession», based on the inclusion of history in technical curricula as a way of «enlarging» the sympathies of students beyond the purely technical. Colleges which train the technical and professional specialists for the construction industry sometimes set aside a small number of hours per semester for «history». Thus, civil engineering students might be given a series of lectures on the development of roads, harbours, water supply etc. from the 18th century to the present date; or quantity surveying students might be asked to read Thompson's *Chartered Surveyors* or Nisbet's *Quantity Surveying in London during the 19th century*.

Much of the history taught on these courses is really about the formation and development of particular professions, whether surveying, civil engineering, architecture, town planning, environmental design and engineering, or landscape gardening. Most of these professions are relatively recent, the earliest dating from the eighteenth century. To restrict building history to the rise of a profession is necessarily to omit the great mass of construction in earlier epochs.

Furthermore, study based on the history of individual professions is inappropriate for the construction industry not only because of the extent of the inter-relationship between professions in this sector, but also (as Ive and McGhie[2] have pointed out) because building is «closely connected with other manufacturing industries and service sectors».

Yet another approach to building history draws its intellectual inspiration from the idea that history has relevance to contemporary society. Morice[3] in his article on the role of history in a civil engineering course gives a two-part justification for the study of building history: it will help engineers design better structures, since building is «littered with examples of (structural) failures which historical insight could probably have avoided»; it will also develop an appreciation of the impact which major engineering works can have on the environment since «historical study will cause the young engineer to be more aware of his public accountability in environmental matters».

Morice's approach raises a number of questions. At the level of historical scholarship it has not been settled that «lessons can be learnt from history», as witness Hegel's famous dictum; some historians argue that the study of history, as the study of human thought and action, is of intrinsic value irrespective of any practical implications. Moreover, in terms of practical performance it is questionable whether history can be used as an aid to better structural design by «professional engineers». Hatchett[4] has shown in a case study that the failure of the King Street bridge in Australia in 1962 was caused by an unholy mix of new technology (the introduction of high tensile steel), failure in communication between consultants and subcontractors, faulty inspection procedures and difficulties in the method of procurement. Such work suggests that trainee engineers should address general questions relating to the current construction industry by focusing on topics such as methods of procurement, management control, labour relations, technological development etc.

Another approach to building history can be found in those graduate/postgraduate courses focused on urban history. The rise of urban history in Britain in the 1960s and 1970s represented an attempt to subsume construction history within the boundaries of the historical study of the city. H.J. Dyos, the doyen of urban historians, bemoaned the lack of detailed research on the operations of builders and developers and considered this form of history to be an important component of studies on the urban fabric. Later writers on urban history have followed this approach. Ravetz,[5] for example, in her book on post-war urban planning gives an excellent description of the way industrialised building techniques (and the forces which created this technology) restructured the urban environment of many cities.

It can be argued, however, that the urban historians' approach to construction history results in an inadequate level of conceptualisation and theoretical perspective to this subject. Despite what Dyos[6] says about the range of research work on the urban past («from the archaeology of urban development in the Dark Ages to community patterns in the modern metropolis, from York in the age of reform to Blackpool in the age of affluence»), most of this work concentrates either on the urban transformation process that began around the year 1800 or on subsequent aspects of urbanisation. Urban history by definition concentrates on the city and gives little attention to the rural aspects of living. What, therefore, do urban historians make of the intensive building activity that occurred in rural England between 1570 and 1670 (what Hoskins called the «Great Rebuilding») that ended the medieval preference for impermanent buildings and resulted in the building and successive rebuilding of permanent vernacular houses? While past civilisations have occasionally been dominated by their urban institutions, throughout historical time most people did not live and work in urban concentrations (even in 1800, when the world's population was estimated to be about 1000 million people, less that 3% could be described as urbanised.[7] This suggests that construction activity was mainly located in rural communities which were geographically diffused and supported a highly localised building industry.

Building history as taught in British universities can be summarised as being about the histories of styles, building types, the rise of the professions, engineering and urbanisation. What is conspicuously missing from these pedagogic approaches is viewing construction as a problem in industrial organisation. Construction throughout historic time has often involved the

mobilisation of thousands of people —either by compulsion or through the market place— and large sums of money (about one-third of church income during the Middle Ages was spent on cathedral building). The organisational problems in erecting buildings, including the supply of materials, construction operations and financial administration, have been mostly ignored. Although the industry satisfied one of man's most basic needs and was, even into the industrial age, one of the largest employers, it hardly finds a place in most surveys of the economic history of Europe, and its historical development is excluded from college courses dealing with building history.

Only a small amount of work has been published which deals with the theoretical problems associated with furnishing the construction industry with an historical identity.[8] One of the surprises in studying construction history is the lack of attention given to the identity and scope of the subject. Despite the mass of literature written on various aspects of construction from an historical viewpoint, there is a marked paucity of works dealing with the conceptual framework and theoretical basis of the subject. Most of the published work on this topic appears in the journal of the Construction History Society.

Why has this been the case? Part of the reason is found in what Ravetz claims is a «deep division in society in which technical and industrial processes and their workers are socially divided from policy makers, academics and professionals . . . » Construction as a manner of production is considered to have little social relevance. Another reason for the lack of attention given to construction history has been the proliferation of historical specialisms (urban, architectural and economic history) which deal with aspects of construction as part of their academic portfolio.

Construction history as a subject of study focuses attention on a variety of important subjects which might not otherwise be taken seriously. It gives historical perspective to the way the built environment was literally pieced together, dealing with matters such as the structure and ways of working of the building industry, the type of technology used, developments in the labour process, the relationship between construction and the development of the economy etc. There is however no intellectually compelling reason for seeing the subject as an independent historical discipline. Most of the work could be located in one or other of the existing specialisms which deal with the history of the built environment, such as urban history, architectural history or the inter-disciplinary approach found in the Bartlett International Summer School. Each of these has the potential to develop the concepts and methods relevant to the study of building activity, provided only that enough interest is taken in this area.

In practice, however, the history of construction has been largely neglected by these specialisms and instead has been the preserve of amateur historians, technical specialists and academics whose commitment to construction as a field of historical study is restricted to the duration of a few research projects. A price is paid for leaving construction history out —urban history, for example, makes little sense unless some attention is given to the way in which the urban fabric was made. Furthermore, the building industry has been of fundamental importance to many economies for long periods of time. This suggests that construction history should not be dismissed as an idle intellectual pastime. Its main vindication as a subject of study (apart from any intrinsic value) is the unfortunate reluctance of the more pertinent historical specialisms to enter into discussion about building production, counterpoised with a recognition of the importance of this activity to socio-economic life in the past.

The establishment of the *Construction History Society* in the UK has helped construction history to emerge as a separate intellectual discipline. The Society was founded in 1982 with the intention of providing a focal point where those interested in the history of construction, historians and people in the construction industry could meet and exchange ideas. The principal aims of the Society are to encourage research into the history of construction and to assist in its dissemination while at the same time locating, identifying and listing primary source material and to encourage its preservation. The Society organises an annual lecture/seminar and publishes a prestigious journal and quarterly newsletter.

A small coterie of staff at South Bank University (London) considered it time that construction —which in some cases had the highest concentrations of labour found in any industry before the industrial revolution— found a place in the academy to study its historical development. South Bank University is ideally placed to offer such a programme. With one of

the largest Built Environment Faculties in Europe it contains staff from a variety of backgrounds (architects, construction managers, conservationists, building economists, structural engineers etc.); while the focus of the Faculty is on the needs of the modern building industry, there is a subculture whereby historical topics (seen as peripheral to the core curriculum) are discussed in an informal way.

The undergraduate programme in the Faculty has two units (the *History of Building Design and Production* and the *History of Building Engineering*) which not only deal with architectural history (when major transformations occurred in historical style) but also offers an insight into how earlier societies organised their production process. For example, students are both introduced to the refinements of Greek architecture (the orders and sculptural treatment of buildings) and the production process that allowed Greek temples, stoas, theatres, stadia etc. to come into being. Many building accounts have survived from classical times that allow interesting questions to be raised about, for example, the economics of temple building in ancient Greece and the technology employed on building projects.

The University has maintained close links with the *Construction History Society* through sponsorship of the annual seminar, hosting conferences, publishing papers in the Society's journal and newsletter, delivering guest lectures, secondment to the committee of the Society and storing archive material.

In recent years the Faculty of the Built Environment has been experiencing declining student numbers, associated with the downturn in the Construction Industry and the difficulty that graduates have in finding work. Also, there has been an oversupply of education courses in construction, with too few students chasing too many courses and, of course, the previous Government had cut funding to the Universities. To offset the decline in graduate numbers, the Faculty has adopted the strategy of developing more postgraduate courses, to attract those in work seeking professional consolidation and development, or those who want to make a career change.

The «traditional» academic view with regard to postgraduate courses is that they should be offered in institutions with a sound research base to support teaching. This strict requirement is difficult to achieve with construction history since the subject area is a focus for a variety of forms of knowledge rather than a form of knowledge itself. Construction history lacks fundamental research that develops models and theories which unify the subject area. Therefore much of the research work associated with construction history while being of direct relevance to the subject area, does not address the core issues involved. Benefits can accrue from this intellectually diffuse approach: the construction historians perspective is positively enriched by not being committed to a single methodology. The lack of agreement as whether to pursue an economic approach based on analysing the influence of building in the economy, or a technological approach that focuses on the various stages of structural development, or an approach that concentrates on the role of labour in the building process, can help to produce a range and variety of research that gives serious consideration to the complex historical processes that have shaped construction.

The Built Environment Faculty at South Bank University is engaged in a variety of research that is of relevance to construction history. This work includes historical studies of speculative builders, housing markets, urban form, building in London and the development of architectural style. It was expected that current research activities together with future projects would underpin teaching on the Master's course. Once the course is established, commercial research contracts would also be sought.

Currently no university in the UK offers a Master's programme in construction history and, as far as is known, no higher education institution is proposing to offer such a course. There are no professors in construction history, no chairs on offer and only limited opportunities to consider construction from an historical viewpoint. This suggests one of two things: that resource constraints have prevented the setting up of such a programme or that there is a limited demand for a course in construction history.

COURSE DETAILS

The design of the MA curriculum was undertaken by a Course Planning Committee following three simple steps. First, the material that is of central importance to construction history was identified. The aim of the

Committee was to avoid serious omissions on the one hand, while guarding against an over comprehensive approach.

Information on what should be included in the curriculum was collected from a number of different sources including the scrutiny of architectural/engineering history curricula offered by other UK universities, by discussion with academics involved in the research and teaching of both mainstream and building history, by questionnaire survey, by a literature search of books, journals, abstracts, research reports etc. that deal with construction history issues. What constitutes the basic knowledge areas of construction history was surprisingly easy to formulate. The organisational problem of erecting a building breaks down into three distinct tasks: supply of materials and labour, construction operations, and financial administration. This material reality faced builders in the past as well as the present. Using this process model as a guide, it becomes relatively straight forward to disaggregate each step of construction into discrete elements that reflect the complexity of building. The Committee formulated a number of basic knowledge areas including: production of building materials; labour process; design and designers; history of structural engineering; organisation of site operations; conditions of work; building contracts; economics of building; construction and the economy; money supply and wages; building craft guilds; mathematics/geometry and construction; industrialisation of building production and history of building legislation.

The second step followed by the Committee was to organise this material into a coherent curriculum containing material essential to the study of construction history and which could form the basis of compulsory areas of study. The Committee felt time and spatial limits should be placed on the curriculum material: essentially the core programme is focused on construction activity in the UK and to a lesser extent Europe. Where it is essential for the understanding of particular topics (i.e. the development of tall buildings) other locations have been considered. Students undertaking their *Dissertation* will most likely use archive material relating to building in the UK and therefore the teaching programme should support and interpret this material.

The need to refer to historical sources means the course is focused on the period between the High Middle Ages and recent times. From the High Middle Ages onwards, the written word survives in greater abundance than any other source for Western history. Both Salzman[9] and Colvin[10] summarise the wealth of medieval public records and accounts that deal with aspects of construction. The fifteenth and sixteenth centuries witnessed not only a marked growth in record-keeping by the state and other corporate bodies, but also the rapid spread of printing which encouraged literate productions of all kinds and transformed its prospects of survival. The best archive material for construction historians dates to the 19th and 20th centuries with a mass of historical sources in written, pictorial and oral form. Also more buildings survive from this period with their associated documentation than any other.

Rather than arrange the material chronologically (i.e. considering building production in the 18th, 19th and 20th centuries), the Committee felt the organising principle should be the building process itself. The central problem of construction from a technical viewpoint is the organisation of supply and labour for a building project: this involves given consideration to the organisational problems of erecting a building, the type of contract employed, the supply of materials and labour. Clearly these topics should form a core element of the curriculum.

The urban expansion of the industrial era brought such a soaring demand for more and better housing and for a greater variety of non-residential buildings and public works projects that in the early 19th century the construction industry was transformed and able to undertake much larger operations, helped by mechanisation. The industrial revolution had such an impact on both demand and technology that it is valid to differentiate analytically the building industry before industrialisation and after. With this in mind, the Committee devoted two core units to building production —one dealing with the industry before the modern period (when relatively little structural change occurred) and the other focused on the modern era which is characterised by quantitative leaps in demand, organisation and technology.

Further revolutionary transformation occurred after the second world war. During the war a small number of contractors were allocated large defence contracts

in the UK would not grant membership on the basis of such a qualification. My experience of modern students is that they are highly focused on careers and earning potential and often choose academic courses that assist these «real» world pursuits.

Academic fashion can also play a part: certain courses at certain times become unpopular. Currently in Britain many universities offering courses in building conservation are having difficulties recruiting students when, until recently, they experienced buoyant numbers.

It may be that construction history is seen as a technical subject —history with the glamour left out— that deals with only a very limited number of narrow questions, and therefore the preserve of the «anorak» or eccentric.

A further possibility is that a new academic subject like construction history needs to be grown and developed. Initially such a specialist area is likely to attract few students; but as these students graduate and disseminate details of the course it will gain a reputation for being an interesting and plausible university subject. Resource constraints at South Bank meant the course was required to recruit a set number of students from its inception. Had a different strategy been adopted whereby the course was allowed time to develop, a more successful outcome may have ensued.

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Gaudí's approach to building

José Luis González Moreno-Navarro
Albert Casals Balagué

Although it may seem trite, it must be said that it is impossible to cover all aspects of Gaudí's approach to building in the brief time allowed by a conference. Given this limitation, certain matters of great interest will necessarily be left untouched. Moreover, the matters that are indeed discussed may not dealt with in the depth that they deserve. Proof of the great extension that would be required for this subject is the book that my partner (González and Casals 2002c) and I have recently published. In that publication, over two hundred pages were required to discuss four of Gaudí's works, but not even that was enough to allow us to present all we know about them.

Now, on beginning this talk, I must make a few remarks to justify this fact for the people who think that Gaudí is simply an extravagant architect, an opinion that arises from a superficial observation of his buildings. It should be emphasized that Gaudí was not only original in his formal aspects —as may be easily seen by looking at his works—, but he was also very original in the way in which he used building resources to arrive at his forms. Consequently, if a mere formal analysis of his work is complex, the combination of such an analysis with the study of his building configurations is even more complicated.

Before starting it should also be mentioned that I do not seek to sing once again the praises of Gaudí. Rather, I wish to discuss him with an admiration and acceptance of what were, unquestionably, his very important innovations, innovations that allow us to credit him with a consummate originality. However, I will do this with a critical attitude that will permit a distinction to be drawn between his extraordinary accomplishments and his enormous mistakes. My primary activity, university teaching, demands that I proceed in this way.

Likewise, on beginning this conference a statement must be made that summarizes in a few brief words the essential facet of Gaudí, the characteristic that underlies his various works and that underlies, within these works, each of the scales of analysis that can be used to study them, from their materials and structures or wrought-iron details to each building as a whole. Does anything of this nature really exist? I can state flatly that it does indeed.

To designate this characteristic, I will be using a word that has already turned up several times in this talk: «originality». Albeit with certain nuances. What's more, there is a phrase that concurs with this, a phrase that is attributed to Gaudí himself: «Originality means returning to the origin». This is a famous phrase that has appeared on posters announcing exhibitions connected with Gaudí in Barcelona in the year 2002, and some architects who admire him even write it on their visiting cards. In my opinion, however, this is a rather redundant and ambiguous affirmation that requires interpretation. To interpret it, we must relate the way that Gaudí acted and the circumstance common to all the phrases that are attributed to him, namely, that we are acquainted with them, in reality, through the memories of his disciples.

My interpretation is that, for Gaudí, originality meant going to root of problems and finding solutions devoid of any stylistic or technical prejudice. If this is called «going to the origin», I am in agreement with the phrase. In short, it would be reasonable to replace this phrase with another one: «Originality arises on finding solutions by the radical analysis of problems». It may be affirmed that this was Gaudí's underlying course of action throughout his whole life and the characteristic of the solutions that he applied to both the smallest elements of his constructions and to his largest buildings.

Of course, this obviously does not mean that this very particular way of proceeding, so characteristic of Gaudí, leads automatically to successful solutions. Solutions may be wrong. Neither does it mean that all solutions are totally innovative. Traditional solutions are correct on many occasions and Gaudí had no difficulty incorporating them into his repertoire. However, it is precisely this attitude of radical analysis that allowed him, on propitious occasions, to act in an original way.

The aspect of propitiousness should never be forgotten, especially in the case of Gaudí. In some jobs he was able to unfold his creativity without restrictions, while in others he encountered impediments. It is only natural to speak more about the former than the latter.

Another prominent characteristic of Gaudí, which was a logical result of the critical attitude that marked him, was the fact that he underwent great changes over the course of his creative life. Generally speaking, people do not have the same inventive capacity on completing their studies as they do after exercising their profession for thirty years. In Gaudí's case, his selfsame critical attitude caused his evolution to be much more intense than that of other persons who accepted the existing criteria in a disciplined way. For this reason, not only will I be referring to the buildings in which he felt free to apply himself, but I will be discussing particularly the ones that are rather more characteristic of his mature age, the buildings in which he was unquestionably able to express himself as he wished. However, this does not mean that clear signs of constructive originality do not appear in his earlier works, such as the Vicens House, El Capricho, the Güell Palace, etc.

His works after 1900 fall into two distinct groups with respect to the concept of creative freedom: an apartment house on a plot of Ildefons Cerdà's Eixample district in Barcelona is not the same as a housing development or two of his churches. Güell Park, the church of the Güell Colony and, by extension, the Sagrada Família, are the works where he unquestionably expressed himself with an almost complete formal and constructive freedom.

Method

Having made these clarifications, which are indispensable in the case of Gaudí, it is now necessary to give a brief description of the method that may be followed to approach his complexity. It is based on the method that I apply in general to both the teaching of construction and to analysis (González, Casals, and Falcones 1997, 2001), involving the interrelationship between two groups of variables arranged by means of a matrix. On the x-axis lie the various scales by which the analysis of a building may be undertaken: the processes used to make the structures and the materials, elements and parts, be they roofs, façades, etc., that give shape to the building which we usually consider really to be defined by the space that it creates. On the y-axis lie the various objectives that have always been pursued by architects: stability against gravitational and horizontal forces, a suitable ambience for its use, and durability with respect to the passing of time, all this through forms and surfaces that create a pleasurable feeling or, in Gaudí's case, a strong sudden emotion in the spectator.

Processes and masonry

With respect to the masonry, materials and, generally, the processes of building, Gaudí used only those that were provided by tradition, that is to say, stone and brick masonry, lime mortars, wood, etc (González and Casals 2002a). He used the major novelty of the 19th century, rolled steel, almost only for a rather modest span in the Güell Palace. Figure 1.(González 1998; González and Casals 2002c, 44–53, 143–155). In the rest of his buildings it always plays a subsidiary role that does not allow the full utilisation of all the possibilities that this extraordinary material provides in relation to spans across spaces. He began to use

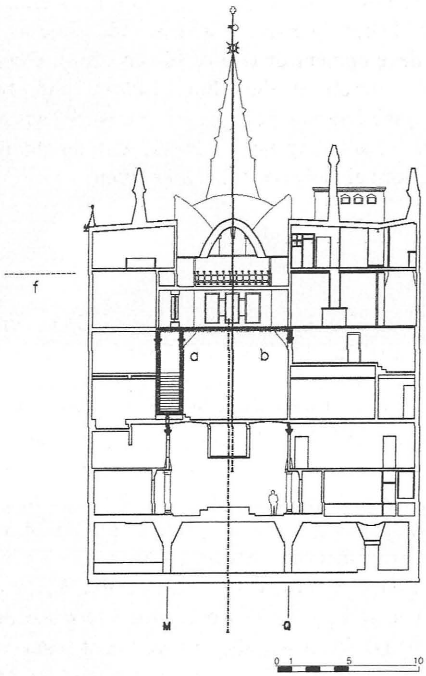

Figure 1
Beam ab, situated over the central space of the Güell Palace, is the only case in which Gaudí used a steel element to cover a span of any considerable size

one of the big novelties of the turn of the century, Portland cement, but always as an unclearly understood substitute for lime in mortars. Lastly, the other great novelty of the turn of the century, reinforced concrete, was absent from Gaudí's work until shortly before the end of his creative life, when he used it in the San Bernabé Tower of the Sagrada Família.

If we search for innovations in his bonds, we find only a very early case: the containment wall opposite the courtyard of El Capricho, where the mason's lines are slanted 45 degrees. In other structures, such as those of rough-stone or dressed-stone masonry, the strictly formal solutions in their bonding are not treated innovatively by Gaudí.

Gaudí also followed a way of doing things of his times and of previous centuries that involved mixed structures: rough-stone masonry with brick, or rough stone with dressed-stone masonry, but he did so in a way that was free of stylistic conditioning factors, seeking maximum effectiveness from a practical standpoint. A notable example of this is to be seen in the Teresian School. Just as Professor Alfonso de Sierra pointed out, the brick masonry of the top floor, with greater tensile strength than rough stone masonry, forms a crown with both expressive-formal functions and structural functions (González and Casals 2002c, 65).

Despite it all, in Gaudí's building processes, this utilisation of traditional structures does not stand in the way of a radical attitude that allowed the architect to introduce major innovations. The most notable case was that of the ruled surfaces he achieved by means of the simple inclination of one of the two rules that direct that generatrix of the surface, that is, the mason's line or string. In this very simple way he went from the plane to what would be one of his great formal and, at the same time, constructive contributions: ruled surfaces. Although one finds examples in early works, such as the hoods of the chimneys of the Güell Palace (González and Casals 2002a, 61), this feature reaches its greatest development in small elements such as the church of the Güell Colony and, with its full power and creative capacity, in the vaults of the nave of the Sagrada Família. This is an innovation that affects the form of the element but not the structure itself. He was able to apply this formal innovation with the greatest force to thin masonry vaults or timbrel vaults (*bóvedas tabicadas*): good examples are the roofs of the pavilions at Güell Park. Figure 2.

Nevertheless, it should be kept in mind that if it is wished to apply a finish that is different from that of the structure itself, these warped forms require claddings that adapt to this surface, which is so different from that of a plane. The achievement of another of Gaudí's key characteristics —colour, with all its possible variations—, led him to the solution of ceramic tiles broken into fragments, which could be used to cover any type of surface. Gaudí used this magnificent surface-cladding and formal solution, which is usually called *trencadís* in Catalan, in a very large number of cases. Figure 2. Although it cannot be said that Gaudí invented it, it may be affirmed that he carried it to really notable extremes, and one case of this is, unquestionably, the finish he applied to history's first hyperbolic paraboloid vaults, in the porch of the Güell Colony.

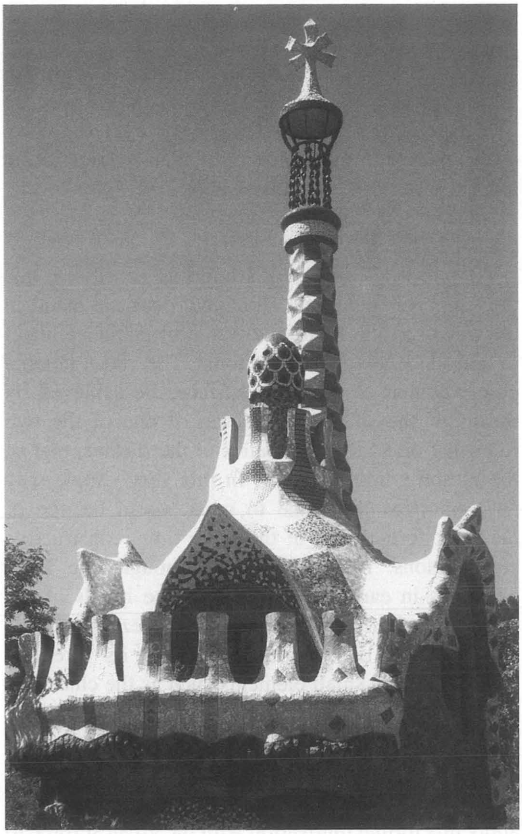

Figure 2
The roofs of the pavilions at the entrance of Güell Park are built with slender ceramic vaults or timber vaults clad with tile fragments or *trencadís*

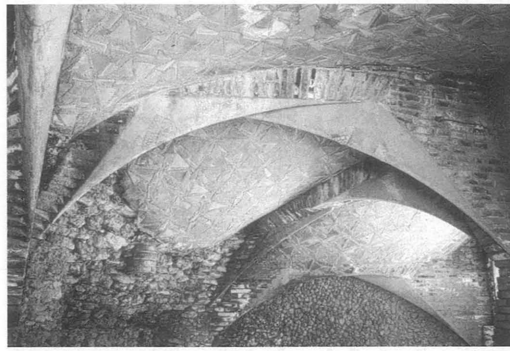

Figure 3
The vaults of the portico at the Güell Colony church are in the shape of hyperbolic paraboloids

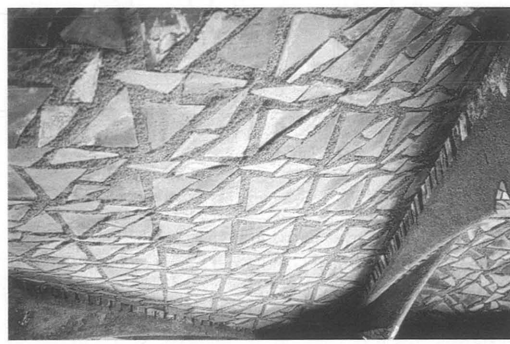

Figure 4
The straight lines marked by the fragments of ceramic tiling concur with the straight generatrices of the hyperbolic paraboloids

ELEMENTS

The vaults of this extraordinary porch are a notable achievement presenting at least three really substantial innovations: a) Their form. Figure 3. b) The way in which this form is clad, accentuating its geometrical features while creating a finish of extraordinary visual quality, Figure 4. c) The building process coherent with the form and its cladding. Figure 5. Just as I have pointed out (González and Casals 2002b; González and Casals 2002c, 175–180), this is a case in which it may be said that Gaudí applies the processes of Roman construction, precisely in quest of the formal innovation entailed by the hyperbolic paraboloid.

These vaults are extremely remarkable, as are the arches that support them, arches that were treated by Gaudí with what could be called a subtle irony. He follows his anti-funicular method while transgressing it since, mechanically, these are straight arches resting on two great brackets; by adding two curved elements, however, they recover the arched form derived from the wire model. Figure 6.

Although the arches in this porch are coherent in their way of working and in their anti-funicular origin, it is more than questionable whether they are coherent with their form and whether the arches of more less parabolic profile that frame the openings in one wall entail a certain mechanical advantage. In a

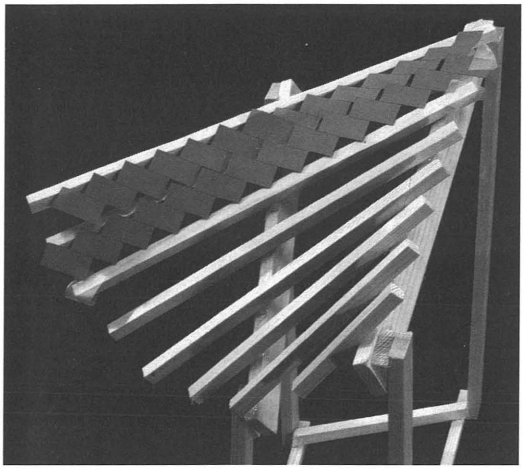

Figure 5
A model illustrating the hypothesis of the vaults' construction using process similar to that of the Roman vaults

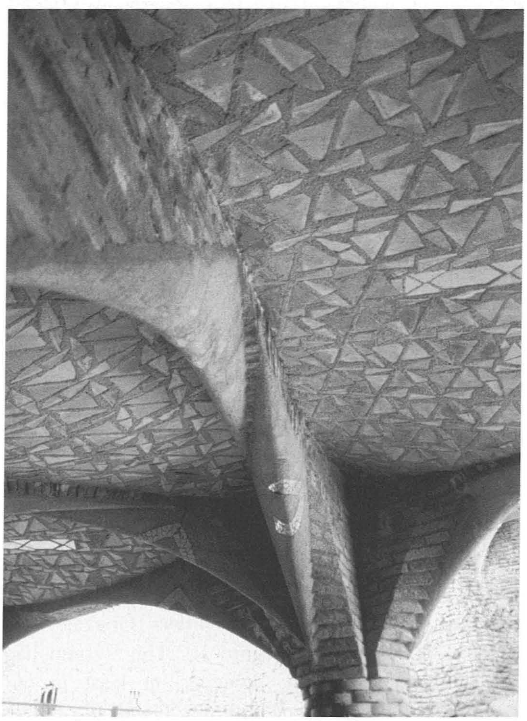

Figure 6
The mechanical arch is formed by three straight sides; its final shape recalls its anti-funicular origin

façade wall, the arch may adopt any form since, even if it departs from a perfect anti-funicular profile, the descent of its loads will find a line of pressures that, issuing from the arch and embedding itself in the masonry structure of the wall, stabilises successfully the imbalance produced by the void of the opening.

This leads to my theory: the anti-funicular method or the application of graphic statics, in the case of Gaudí, is not so much a matter of mechanical or constructive rationality as a will to create form.

Remaining on the level of elements, two extreme and quite distinct cases should be emphasized that seem significant with respect to the architect's characteristics. One of these, from the standpoint of stability, is the big mistake of the foundation elements of the Casa de los Botines building, which reveals a Gaudí who was most likely acting with an unjustified wilfulness (González and Casals 2002c, 155–161). Figure 7. As opposed to this, almost simultaneously in time, there is another element that shows Gaudí's concern for users, for the comfort of the people who

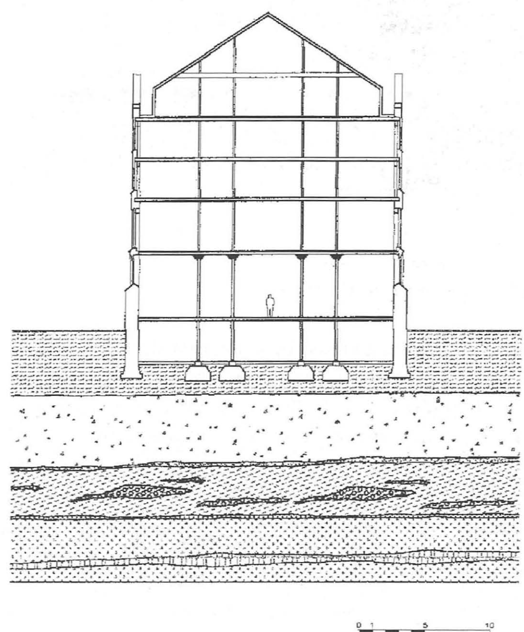

Figure 7
There was resistant soil one metre beneath the point where Gaudí situated the foundation plane of the Casa de los Botines

lived in his buildings: the wooden floor assemblies with an unusual cross-section that separate the servants' floors from that of the owners in the Güell Palace(González and Casals 2002c, 114–118). Figure 8.

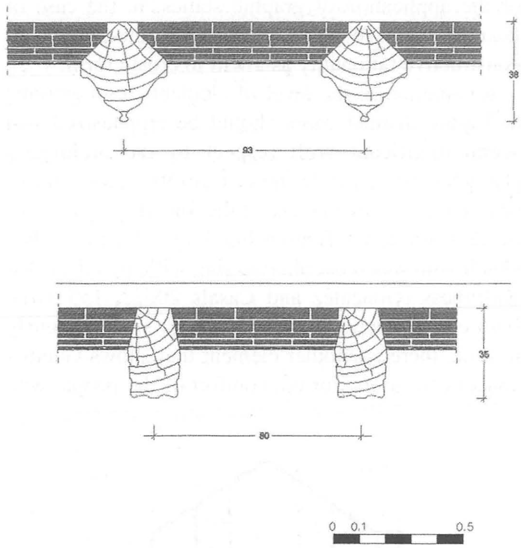

Figure 8
Detail of the cross-section of the Güell Palace's floor assemblies, situated on line ff of Figure 1

Parts of the buildings

Continuing the increase of scale, two significant cases may be pointed out. On the one hand, I can mention the hypothesis of a Gaudí who was excessively acquiescent with his customers by designing a roof that provided them one more floor than was permitted by the ordinances, as it is reasonable to surmise with respect to the Casa de los Botines (González and Casals 2002c, 129–132). As opposed to this, there is a façade that, presenting no disadvantages for users and perhaps even certain advantages —at least during its first one hundred years —, responds to an idea of absolute freedom: this is the façade of the Milà House. Figure 9. It is difficult to find this extraordinary formal and expressive wealth, based fundamentally on its undulating forms alone, but its

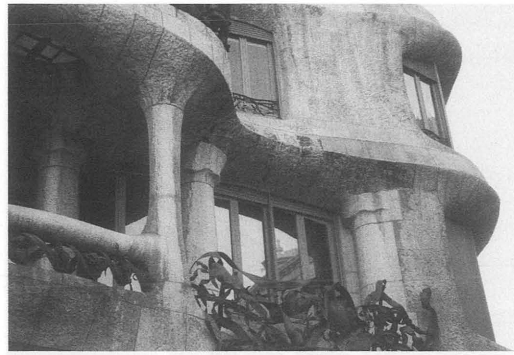

Figure 9
The dressed stone blocks of the Milà House's façade are suspended from the beams of the floor assemblies

consequence is a serious limitation in time. If it is wished to lend greater durability to this façade, its restoration may involve extremely high costs since the iron rods that Gaudí installed, which have already rusted in many cases, must be replaced with stainless steel elements (González and Casals 2002a, 63).

The buildings and their space

In the buildings in their totality, we continue to find proposals of extraordinary originality that are not free, however, of contradictions and paradoxes.

Continuing with the case I have just mentioned, that of the Milà House, this unlimited freedom that Gaudí looked for in the façade was also sought, naturally enough, in the ensemble of the building and in its distribution, for which reason the architect avoided load-bearing walls from the start. To achieve this, Gaudí adopted the porticoed system of columns and girders, precedents of which appear, in reality, only in industrial buildings, of which there were hundreds in Catalonia, derived from the English factories and designed on the basis of cast-iron columns and wooden or metal girders (González and Casals 2002c, 44–45). Figure 10. This system had never been transposed, however, at least in our country, to an apartment building. Gaudí did not so much seek flexibility of use throughout time, as would later be the paradigm of Le Corbusier's flexible plan, as absolute freedom in making vertical

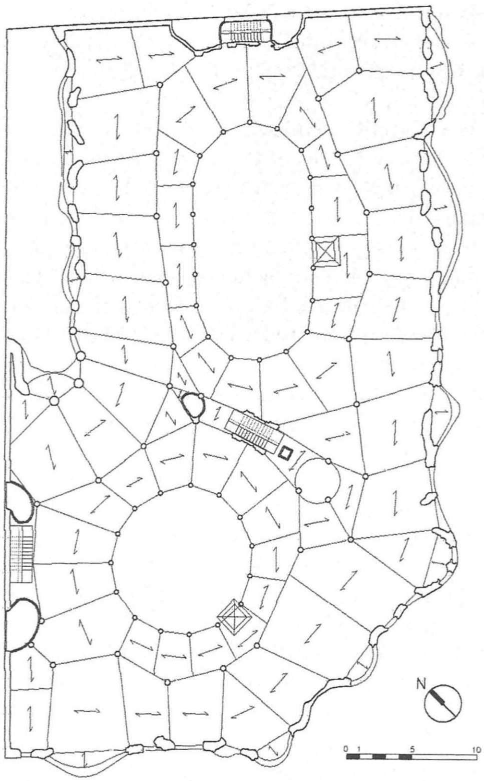

Figure 10
Plan of the Milà House's load-bearing structure

divisions and, consequently, domestic micro-spaces. This did not prevent him from emphasizing that this flexibility of use would have allowed, as he himself stated, the transformation of La Pedrera into a hotel, or as has now happened, the transformation of some of its floors into exhibition halls

If we continue to consider apartment buildings but go on to the y-axis of our analysis model, with its ambience factors, it is indispensable to mention the extraordinary case of the Batlló House, which is also exceptional for other reasons. But here I will quote what was written by the expert on the building and its restorer, Josep Maria Botey: «It is exciting to discover the passive ventilation systems that Gaudí applied on connecting the courtyard to the main façade by means of tubes that promote vertical ventilation, boosted by the creation of a laminar current on the roof's skylight, which produces a Venturi effect and, consequently, an absorption and a continuous supply of fresh air, enhancing the typical ventilation system of the little courtyards that characterise the Eixample». Figure 11. (Conference of the architect J. M. Botey in ETSAB, Architectural School of Barcelona in November 2002. Not publied). Botey went on to point out how Gaudí used mechanisms and devices that affect the windows and doors to distribute successfully this ventilation throughout the ensemble of interior rooms. This building has, unquestionably, been studied little or not at all from this standpoint, as has previously been the case with so many other aspects of Gaudí, whose ambient applications have been almost always overlooked.

Now, proceeding still further with this ascending scale of freedom, of size, and of Gaudí's age, it is necessary to discuss his two churches. In his masterpiece we find yet another paradox. The commission was made to him when he was, in my opinion, too young —31 years of age—, and it was moreover an «envenomed» commission since it involved nothing more than the continuation of what another architect, who was very much more conventional than he himself, had already begun. In fact, Gaudí took a long time to contribute something that was really coherent with his creative capacity. All experts have emphasized that the major formal

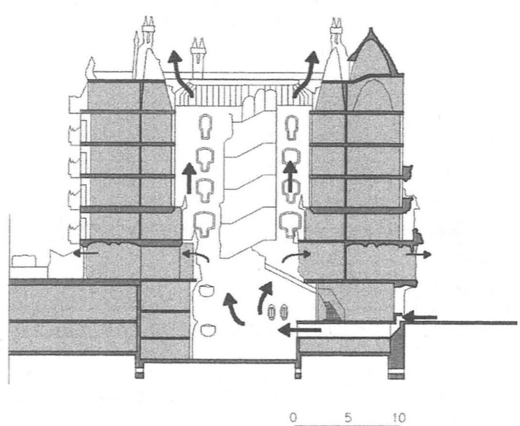

Figure 11
The arrows mark the path of the air flow that Gaudí devised to force the ventilation of the Batlló House's courtyards

contributions of the Sagrada Família appeared from the year 1915 and were the result of Gaudí's work on his other church, which was not subject to practically any functional or, in principle, financial limitation: the church that was commissioned to the architect by Eusebi Güell for the Colony Güell. This was where Gaudí, probably for the first time in his life, was able to do what he wanted in an unhampered way: to make a physical reality of his system of devising forms by means of the anti-funicular process. In the passage from wires to reality, he brought to bear all his constructive-formal creativity. See figures 3 and 6.

Nevertheless, if we make an analysis from the standpoint of adaptation to the always scarce available resources, we can demonstrate that the space resulting from this system is not free of serious problems with respect to spans, internal views and, especially, building processes. Figure 12.

All this reaches its high point precisely in Gaudí's great work, the Sagrada Família. Although the determination of form here does not follow the anti-funicular method, it does make use of graphic statics, which is nothing other than a similar and more economical way of achieving the same end: to find linear elements of structure, both straight and arched, that receive only axial compression stresses and that

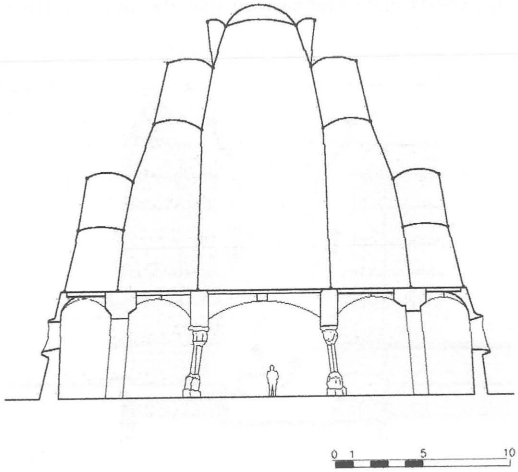

Figure 12
Schematic of part of the Güell Colony church. The upper part, which was never built, is represented by the wires of the funicular model

consequently do not require auxiliary elements such as buttresses or, in the case of the Gothic style, flying buttresses (González and Casals 2002c, 82–95, 167–170).

This objective dominates the design process of the towers and the aisles of the Sagrada Família church, as is reflected in many of the maxims recorded by Gaudí's disciples.

In the case of the towers, the surpassing of the Gothic lay in making them continuous by means of their spindle-shaped form, far from the caricature drawn by Gaudí himself when he stated that Gothic towers were like spyglasses, reducing their cross-section by setting back each successive floor. Gaudí's form is obviously different, but it would be difficult to prove that it entails any mechanical improvement. Having reached the point where we now stand today, there is something ridiculous about criticising the Gothic towers and it suffices to recall in this respect the towers of Bologna, without mentioning those of the French, English or German cathedrals.

Another aspect in which paradoxes continue to pursue the works of Gaudí is the design process he used for the central nave. The quest to surpass the Gothic led him to make statements that were, in the end, rather grotesque, such as when he said that the Gothic flying buttresses are something like crutches. That was the old trick of seeking to outdo someone by mocking him (Schopenhauer [1864] 1990). However, just as may be easily seen from the accompanying drawings, in order to balance his aisles, since the structure was made of stone masonry (resistant to compression alone), Gaudí had no choice but to resort to another Gothic technique, which was none other than the use of pinnacles that verticalise loads, carrying them to extremes in hyper-pinnacles that, in the central nave, become another building that reaches the height of a second superimposed church, comparable to the cathedral of the city of Barcelona itself. Figure 13.

If it is wished to avoid the use of buttresses, then the anti-funicular system or graphic statics applied to single-resistance structures do not offer any other solution than this one, which calls for the reduction of spans at the same time. This is one of the key characteristics of the Sagrada Família: compared with those of many Gothic buildings, its spans may be considered even ridiculous. This may be observed by comparing it, for example, to Bologna's San Petronio

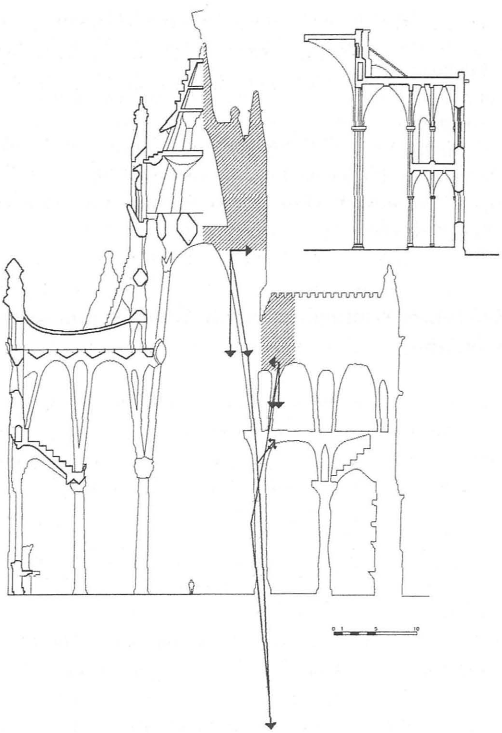

Figure 13
On the left appears a half section of the Sagrada Família's nave and on the right is a diagram of gravitational forces. The shaded areas indicate the parts of the building where, due to their weight, the resultant would be as vertical as possible. At the upper right is shown the cross-section of Barcelona's Cathedral, which has a height similar to shaded tall body of the Sagrada Familia

where a single column allows the coverage of 350 square metres. For this same surface area, Gaudí needs four and a half columns. The fact that the Sagrada Familia is now being continued by means of a reinforced-concrete structure only goes to highlight this enormous paradox of a system that seeks to surpass the Gothic but cannot do so and requires a building method that is so far removed from it as is this characteristic technique of the 20th century.

Unquestionably, Gaudí's method does not even come close to surpassing the Gothic system, although this does not mean that it is either better or worse. It is, in any case, extraordinarily different. This is a key aspect. Gaudí did not surpass the Gothic but he was an unquestionable genius who created new forms in history, forms that are valid in themselves and coherent with the aesthetic revolution in which he lived in the last part of his life. He displayed an extreme creativity based on the consummately intelligent use of the geometrical properties of ruled surfaces. This is the great legacy that has been left to us in the aisles of the Sagrada Família. Figure 14.

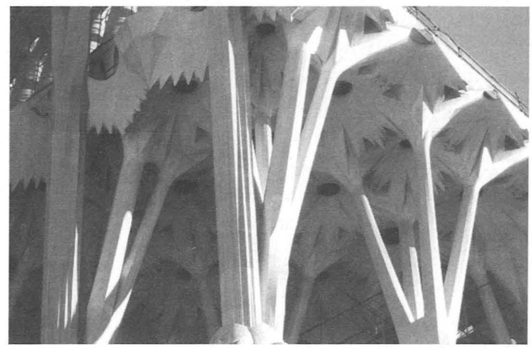

Figure 14
Despite all its contradictions, the ensemble of vaults of the Sagrada Família's nave is one of Gaudí's most beautiful works

Form and substance

If we seek an identifying characteristic of all the contributions of Gaudí's last stage, it has already been stated on a multitude of occasions that it is none other than formal continuity, justified by Gaudí many times for constructive reasons, in most cases with insubstantial arguments. These are solutions that only make sense, in reality, from an aesthetic-formal standpoint, which is neither good nor bad but forms a distinct architectural reason that must be valued as such.

For me, however, continuity is a characteristic that actually goes beyond the merely formal since it extends to the continuity of scale, the continuity of arguments and the continuity of objectives (González and Casals 2002c, 194–197). Gaudí —and in this respect he succeeded Viollet-le-Duc— is the architect who stands farthest from the fragmentary theory of academic architecture, the origin of which lies in the

erroneous interpretation of Vitruvius's triad, which continues unfortunately to be upheld in all the world's architecture schools (González 1993, Casals 2002). For Gaudí, architecture requires and takes advantage of the assemblage of all scales, and of the assemblage of all objectives and of all the intricate relationships that may be established between them.

This is indeed the great architectural-constructive legacy that makes Gaudí —divested of the useless character of a legend and with all his contradictions fearlessly laid bare— deserving of consideration as a great architect who offers lessons on methods for carrying out the quest for roots — for the origin, as he said. These are methods that allow us, on the one hand, to approach problems from their beginning and, on the other, to develop, without ever marginalising users, solutions free of all stylistic and constructive prejudice, even if they are not devoid of the risk of error, just as Gaudí showed us throughout his career.

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History of construction: An estimable resource in the actual crisis of civil engineering?

Werner Lorenz

Rise and Decline —A Five-Minute History of the Civil Engineer

In the year 1762, the librarian of the Roman cardinal Albani, Johann Joachim Winckelmann, born in the North German town of Stendal, published a pamphlet titled «Anmerkungen über die Baukunst der alten Tempel zu Girgenti in Sizilien» (Remarks on the Architecture of the Old Temples at Girgenti in Sicily) (Winckelmann 1762). In no time, this publication became the manifesto of the young neo-classicist movement in Europe. Evaluating his own systematic research of antique architecture, Winckelmann calls it the most appropriate model for any form of architecture, including contemporary. He distinguishes clearly between the «Wesentliche» (essential) and the «Zierlichkeit in der Baukunst» (daintiness of architecture). The clear distinction signifies an abrupt turning away from the previous baroque perception of architecture. The concept of the «essential» introduces construction as a defining parameter into architectural theory. According to Winckelmann architecture results primarily from constructive considerations.

Noteworthy also is the context of his publication. Only a few years prior, in an Italian publication from 1748, one can find the term «inginiero civile» and in 1768, the term civil engineer is used for the first time in England, where the first «Society of Civil Engineers of the Kingdom» is founded in 1771. The civil engineer is born. (Schimank 1939; Woodley 1999)

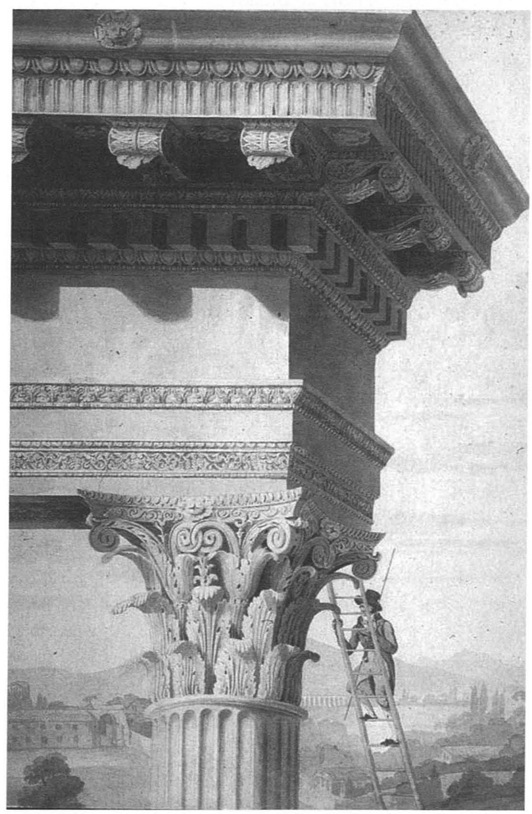

Figure 1
Student measuring the temple of Castor and Pollux in Rome, Henry Parke, 1819

Seventy-five years later Europe has weathered the first storm of industrialization. Quite differently and in much bigger dimensions than foreseen by Winckelmann, the revolution of all social life brought about by production and labor promoted architecture's technical ties. With the new materials of the iron century, the technical and ingenious aspects take the center stage of architecture and along with them, so does the structural engineer —although the term engineer is still not used and is subsumed by the term «architect». The new self-esteem of the engineer-architect is expressed vividly in a song composed especially for the convention of German architects in the town of Halberstadt in 1845:

Wer bahnt dem Fuße sichere Wege?
Wer zwingt den Strom, wer schützt den Strand?
Wer legt dem Fortschritt Eisenstege?
Wer bändiget der Städte Brand?
Wo Wogen stürmen, Flamme leckt
da hilft der kühne Architekt!
(Zeitschrift für praktische Baukunst 1845)

(Who guards the foot across the ditches?
Who forces rivers, shields the shores?
To progress offers iron bridges?
The city's fire tames and moors?
When breakers storm and flames do lick
Help comes from the bold architects!)

An additional forty-four years later, in 1889, two steel constructions of previously unimaginable dimensions, Tour Eiffel and the Galerie des Machines, are presented to stunned visitors at the Paris World Fair, which was held in commemoration of the 100th anniversary of the French Revolution. At the same time in Scotland, the Forth Bridge nears its completion. All the world talks about the work of the engineers, the tower of a height of one thousand feet, the hall that is more than 100 meters long and the bridge that spans half a kilometer without support. With every new record the esteem of the civil engineer rises. Hardly anyone can deny the fascination of the engineering products. The architect Henry van de Velde memorably summarizes the public reception of the engineer in his criticism of the Galerie des Machines: «These artists, the creators of a new architecture, are the engineers.» (Giedeon [1941] 1978, 157; Lorenz 1989).

One hundred years later in 1989, Dichter-Institute of Zurich conducts an opinion poll among young school graduates and soon-to-be civil engineers, which is soon after publicized in the Swiss trade magazine «Schweizer Ingenieur und Architekt» (Kiener 1991). Half of the graduates who were asked classified the profession civil engineer as «out». Sixty percent of the young civil engineers deem themselves in public opinion primarily as «calculating menials» of architects. Eighty-seven percent expected to be regarded as «destroyers of nature». No more song, no more praise. The bold and innovative hero has turned into a frustrated and at best diligent administrator of an environmentally damaging and ugly infrastructure.

This five-minute history of the civil engineer and its social projection evokes far more than a sense of unease. It tells about the gain and loss of a fascination that nowadays appears alien. It is a story of the radical decline in the perception of a profession.

It is quite remarkable that in only a few decades engineers apparently managed to effectively gamble away the enormous capital of acceptance amassed by our predecessors over two centuries. Suddenly nothing less than the disappearance of the civil engineer is at stake.

That does not mean that civil engineers will not be around any longer. We will rely further on their calculations, use their technical extensions, glide elegantly across their bridges and take off from their airports. At stake is something different, at stake is the civil engineer's loss of significance in construction, the loss of their inherent culture-forming role for the built environment and its public reception. At stake is the dissolution of the engineer's profession into the meaninglessness of a technician's job.

However, it is even more remarkable how we react to this. Even though it has quite clearly been on the horizon, even though there have been admonishers and no lack of appeals to re-orientation, engineers have not addressed the problem courageously or «ingeniously». Rather they have demonstrated the inertia and stubbornness of a giant tanker in their day-to-day practice as well as in training. They continue as before in a speechless mix of resignation and spite, interpretation of regulations and blindness.

The diagnosis is clear. Civil engineering is in an elementary crisis. The direct result is the often-mourned loss of a culture of construction; the indirect result is the disappearance of the engineer. It is not difficult to reinforce the diagnosis with numerous other observations.

What has happened? In my opinion, two aspects of this crisis deserve special attention:

One aspect is, we are neglecting elementary engineering virtues, which have been developed and cultivated for centuries. Instead of adopting and transforming them for the future, we have forgotten about them.

The other aspect is, we have forgotten what it means to take responsibility.

VIRTUES

We have become skeptical towards «virtue». The word not only seems too old-fashioned; it appears too simple and pure to be used by us. Other terms have long taken its place.

One of the terms is the concept of the guiding image or «Leitbild». However, what images guide an engineer? Is it safety, speed, reliability in scheduling, effectiveness and a high level of competency? For instance, when consulting the publications of the main organization of the German building industry, one finds wordy information on today's civil engineering requirements (Wirtschaftsvereinigung Bauindustrie 2000). A civil engineer should be a competent, efficient and reliable partner, be able to work in a team, have the readiness for interdisciplinary cooperation as well as creativity, imagination and the power to lead other people and, most of all, one is expected to think holistically and act as a generalist.

That is all somehow right; nevertheless, the words quickly grow cumbersome, and where is that which is special about a civil engineer?

Let us dare to speak of «virtues». The dictionary defines them e.g. «as ideal types and images of personal excellence». According to the philosopher Hans Jonas, virtues project «the best possible being of human beings». (Jonas 1979) Virtues are smaller, more modest than large «guiding images» and wishful requirements. They stand in the second row yet they are more direct, concrete and simpler. Perhaps I like the term virtue because it alludes to tradition and something old. Virtue is directed not only towards future but also to origin.

We are familiar with common human virtues such as courage, consideration, moderation, wisdom and justice, but how about the special virtues of the civil engineer? Only a few will be listed here in quick succession. They may also be interpreted as «attitudes for constructing» attitudes that have become rather rare today.

SIMPLICITY

Simplicity in this context implies the greatest simplicity possible as a primary criterion for optimization. Simplicity of approach should be regarded highly especially today when a sophisticated calculation technique tends to seduce us into believing we can somehow calculate everything. The best among the engineers have always known about simplicity. For instance, the ingenious pioneer of building with reinforced and pre-stressed concrete during the first half of the 20th century, Eugène Freyssinet, schooled at the École Polytechnique, one of France's elite schools, always emphasized that a

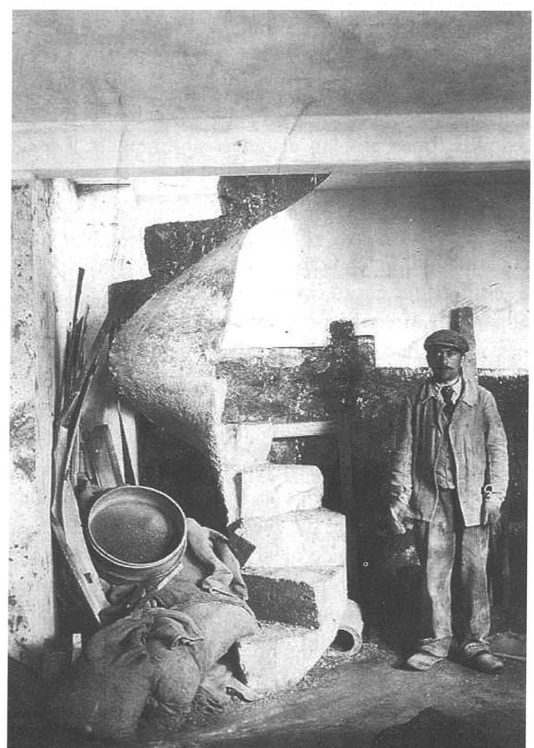

Figure 2
Spiral staircase, Francois Hennebique, about 1900

attitudes and approaches make me think of one who had to understand something about constructing even if he decided in the middle of his life to change from engineer to poet. I am talking of Heinrich Seidel the Prussian designer who was responsible for the construction of Anhalter Station in Berlin. Seidel, once vividly summarized his perception of construction by saying: «Konstruieren ist Dichten!» (To construct is to write poetry). (Mülder 1997, 77)The work, the construction perceived as an artful fabric with all virtues woven in —this image may serve well as an ideal of the engineer's work.

History is indispensable in order to weave such a fabric.

Responsibility

When speaking about responsibility, engineers think about the immediate responsibility of the architect for the secure technical success of the building. We assume that this is old hat and that responsibility has always meant liability. Maybe we even come across the codex Hammurabi, the famous cuneiform writing from Babylonia times which codified rigid punishments for breach of contract in the building business: if a house collapses and kills the son of the owner, the builder has to sacrifice his son's life etc. Then we look at our engineering contract and feel certain that our liability is a little less stringent but other than that, the thing about liability has remained the same.

Has it really?

It appears to me that taking responsibility had a different air about it at other times. It felt different.

Let's take only the 19th century: to be an engineer in the time of early industrialization; to construct with previously almost unknown materials —it was a fascinatingly open era. One builds into a vacuum of material science, measuring theories, technical rules, regulations and norms. None of that exists. Instead, there is a spirit of departure, courage, delight and cunning and the prospect of great transactions. The engineer is liable for the success of his work from head to toe, often with an immediate financial involvement in his projects.

The list of respective heroes is long. George Stephenson's quote, «I do not know yet how to but I can tell you I will do it» (Ricken 1994) is as characteristic as the tragic family story of the Roeblings. The father John had to die in an accident at «his» East River Bridge before bequeathing the task to his son Washington A. Roebling, who himself paid with life-long paralysis for his restless immersion into the murderous labor conditions of the Caisson foundation (Steinman 1957; Farrington 1993 [1881]).

To be liable with body and soul for one's work —hardly anyone presented this attitude as fully as the icon of spirit of British engineering, Isambard Kingdom Brunel. He allowed not even an inch of personal distance from his buildings and took almost physical responsibility for success or failure of his at times, seemingly whimsical buildings. It is no surprise that Brunel's ethic of responsibility made him an enemy of standardization. In Brunel's words: «No man, however bold or however high he may stand in his profession, can resist the benumbing effect of rules laid down by authority.» (Rolt 1989, 283)

Brunel was a uniquely fascinating person and yet an emblem of a whole century. Robert Thorne titled a lecture about him «The engineer as a hero» (Thorne 1999), and recentlythe German poet Hans Magnus Enzensberger dedicated a poem to him:

> Jede Katastrophe ein Sieg, jeder Sieg eine Katastrophe.
> Soviel Energie hat nur ein Ertrinkender (. . .)

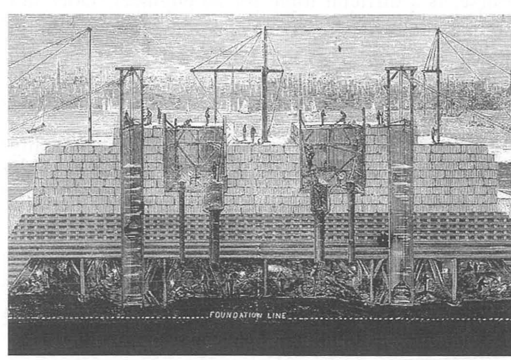

Figure 5
New York, East River Bridge, workers in the caisson, about 1880

Der große Ingenieur, klein von Gestalt: Ein Nervenriese.
Manischer Frühaufsteher, 50 Zigarren am Tag.
Von einem Projekt zum andern jagte er in seiner schwarzen Brischka, stieg aus, melancholisch, ein Zerstörer, der Vergils Eklogen liebte, und schrie: Ich kann niemand brauchen, der mir dreinredet. Ich brauche Werkzeuge. (Enzensberger 1977, 73)

(Every catastrophe a victory, every victory a catastrophe
Only a drowning man has this much energy (. . .).
The great engineer, of small build: A giant of nerves.
A manic early riser, fifty cigars a day;
Chasing from one project to another;
Melancholic, a destroyer who loved Vergil's Ecloga and shouted:
I cannot use anyone who tells me what to do. I need tools)

We have said good-bye to heroes a long time ago, haven't we? Deviation and not injustice marks the crossing of borders in our standardized world. The responsibility for the success of one's own technical work has been reduced to a question of insurance. Do we really believe that such a climate of irresponsible liabilities could move the young men and women we wish for because of their courage, involvement, accuracy and creativity to become civil engineers? No, responsibility then and now is not the same. The word remains but the content has changed.

Beyond liability, another aspect of responsibility has to be considered. We cannot get around defining the term in the sense of responsabilitè morale, as did de Lalande —the responsibility, in the sense of the duty of humans as reasonable beings, to confront the positive or negative evaluation of our deeds. The philosopher Hans Jonas dedicated himself to this topic in his writing «Das Prinzip Verantwortung» (The Responsibility Principle) (Jonas 1979). Explicitly he pointed out that today virtues alone are no longer sufficient. Precisely because our present deeds cast shadows into the future that are longer than ever before, we require a far-reaching principle, directed towards the future.

Until the modern age every construction tightly conformed to an ethical context. The medieval planner and designer fulfilled his task nearly anonymously. In the 6th century the emperor Justinian decided that no name should be attached to the outside of a building other than the emperor's or the name of the person who paid for the building (Ricken 1994, 21). Only rarely, we find a hastily chiseled portrait of a cathedral builder hidden under the pulpit, in the apex of an arch or at the edge of a pillar. His responsibility was subsumed in the group's system of values. The tasks, the goals, the rhythms were quite clear as required by the era.

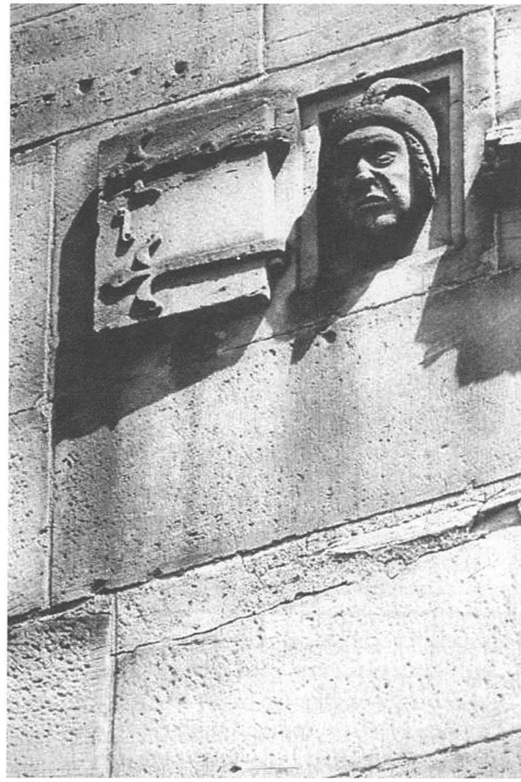

Figure 6
Schmalkalden, St. Georg, anonymous master builder, about 1500

This anonymity changed in the Renaissance. Is it chance that this change coincided with the birth of the engineer? In 1698, Christoph Weigel published his renowned engraving the «Ingenieur». It illustrates clearly the whole tension of the change which had occurred by then: the greater the pride, the more the once neatly joined goal begins to blur and the attached commentary is tantamount to an intense warning: «Was hilft die Städte messen, und Gottes Stadt vergessen?» (What good is measuring cities when forgetting the city of God?) (Weigel 1698, pl.7)

credible answers to the question of responsibility of today's engineer; only if we can find the courage not to help young students to understand only the mysteries of load bearing, composite materials, soil mechanics or construction management; and only if we —beyond all fatigue- and life span projections— allow and satisfy their hunger for infinity; only then will we succeed in becoming again architects of the future. Only then, can we succeed in preventing the disappearance of the engineer.

As is the case with every serious revision this implies the willingness to question everything, our seemingly self-evident paradigms as well as our seemingly self-evident practices. This implies the unprejudiced question about the lasting quality of our buildings, which has more to do with sustainability than with quality guarantees. This implies the courage to say no and it also implies the uncomfortable thought that despite all knowledge and successes we may not have the best technology of all times at our command and we may not be the best engineers of all times.

Suddenly we are free and open for simple virtues and lived responsibility and suddenly the initially mentioned big-sounding guiding principles for future engineers make sense. From the knowledge at our command to the knowledge of orientation, . . . from linear to holistic thinking, from specialist to generalist, . . . from technocrat to becoming a sensitive engineer.

Leon Battista Alberti comes to mind, the legendary uomo universale of the Renaissance about whose far-reaching interests and abilities wondrous things have been reported. He was not only an architect and author of «De Re Aedificatoria» but also a mathematician, physicist and jurist. A very sensitive as well as successful person: the view of splendid trees makes him cry and his imperative is, «humans can do everything if only they want to». Maybe Alberti's most noble virtue lies in his playfulness. In the depth of an antique bookstore, I recently came across a book wondrously titled, «The Existential Pleasures of Engineering» (Florman 1976). Yes indeed, the pleasure of being an engineer! Traces will be left only by those who build with their hearts.

To sum it up, this means not more and not less than to define ingenious building again and always anew as a cultural task and to define us, the civil engineers, as the proper elite responsible for it.

Without knowledge and awareness of history, this process will fail. History is the key. Leo von Klenze, the famous Bavarian architect complained in the middle of the nineteenth century that the facelessness of contemporary architecture was a consequence of the absence of history from conscience. Probably he was right and probably this applies not only to architecture but also to civil engineering.

Some weeks ago, I found a saying engraved in a beam of an old half-timbered house in Mecklenburg, a little north of Berlin, written in Low German: «Man möt vont Olle liern, nieges tau maken» —To make new things, first you have to learn from the old!

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influences. Constructional events therefore cannot be studied in isolation from their human context, they must be seen as part of a continuum of societal development. The historian studying this phenomenon has to allow for different perspectives on the relative position that technology occupies in the general scheme of things: the conception of what constitutes «technique» at various points in history is inevitably different and it becomes one of the defining attributes of an era.[1] It is generally accepted, for example, that there are profound shifts, in attitudes to as well as application of technology between the pre-industrial and industrial societies, and it is beginning to look like we are entering another phase through the introduction of digital technology. These movements occur over long periods of time and, within particular influence spheres that are circumscribed by prevailing networks of communication. Ideas and practices are adopted according to local circumstance, but their intrinsic character is shaped by external influences as well as internal conditions.

It follows that, in order to get a balanced and «multi-dimensional» understanding of the development of building technology during a historical period —say for instance post-Renaissance/pre-Industrial Europe— in addition to gaining a thorough grasp of the instrumental nature of the field within its local contexts, one has to familiarize oneself with the full range of relevant external contacts to which people from this era were exposed to, understand how these relations worked in practice and what place they occupied in the minds of contemporaries, as well as establish the chronological sequence of events. Given the complexity of this revolutionary phase in European history and the interactive nature of social relations, the paucity and the wide geographical distribution of the source material on technical subjects, the range of languages and sub-cultures involved, this task seems beyond the reach of most individual scholars other than for narrowly drawn specialist topic areas. These complications helps to explain why there are as yet no comprehensive overviews of major developmental themes in construction history across cultural and language boundaries for the period in question.[2] This may be due to the relative novelty of the field of study; in time, it can be argued, such works will emerge naturally from the slow incremental build-up of a knowledge base by scholars working independently. However, the recruitment of sufficient scholarly talent and skill is likely to remain a problem for a subject as obscure and technical as this one and, considering the apparent reluctance of technological historians to engage with the narrative tradition of humanities research, the prospects of such over-arching themes being addressed in the immediately foreseeable future are not good, that is, if it were left solely to the vagaries of ad-hoc individual enterprise.

Construction history needs its grander narratives and general surveys if it is to be accepted as an academic subject of the first rank, written by people with an insider's grasp of relevant internal detail and its relation to the whole, but who can also relate the topic to broader external themes. Others do not seem to shy away from taking such wider perspectives in areas of research closely related to ours, a recent example being, James A Farr's *Artisans in Europe, 1300–1914* (Farr 2000). If they want academic recognition construction historians too have to become bolder in their approach to the subject-area in order to gain a higher profile, and participate as full-blown «construction historians» in academic debates, not as exponents of other disciplines who happens to study historic building construction, amongst other things. Ultimately, however, this subject status will not be achieved simply through the action of individual scholars working on their own. An «umbrella organization» of some sort is required that could act as a champion for the subject and provide a platform for cooperation and debate - one that rests on a sound knowledge-base which, as befits the nature of its subject material, transcends national/ cultural/ linguistic boundaries, as well the tendency towards insular specialization prevalent within the building world.

The, *Call for Papers* for this conference quite rightly draws attention to the fact that much of the groundwork for creating this knowledge-base has already been done by specialists from a variety of disciplines and nationalities. However, this information has to be collated, tested for veracity and disseminated. Remaining areas of ignorance need to be identified and researched so as to establish a comprehensive reference base. It is not feasible to rely on individual enterprise for this task; only an organized group of people operating according to an agreed code and with agreed objectives can manage

such a long-term undertaking. An over-arching framework for communication and action, once set up, could channel resources more effectively and thus ensure continuing collaboration in the future. Not only does an association of like-minded scholars, working together to further a cause significantly improve the chances of a subject-area being take seriously by the international academic community at large, it also increases the scope for more ambitious undertakings. For example, one of the most intractable problems that the construction historian, studying a particular historical event in one locality, faces is how to gain access and insight into parallel developments that took place elsewhere, so as to determine cause and effect relationships across borders. Very few scholars individually have all the resources required to work freely across relevant geographical, linguistic and subject boundaries and, consequently, the progress of research into the subject-area has often been curtailed in the past. An international grouping of scholars could facilitate such cross-border comparative research projects relatively easily, thereby producing a more complete picture of historic developments as well as enhancing our capacity for understanding the nature of building as a universal human activity.

So far I have concentrated on the intellectual argument for establishing a permanent framework for international cooperation in the promotion of construction history as a field of study, because it seems to be the decisive factor. The point I wanted to make is that it is not only more interesting to explore the subject from this angle, its very nature also seems to demand a broader perspective in order to be properly understood. There are, of course, other social and cultural reasons for supporting such a move, but these are too obvious to require elaboration. As for the manner in which a «supra-organization» or international framework like this might be created, the two main alternative routes appear to be: a «fast-track» solution involving the immediate establishment of a centralized body with a home-base or headquarters, a clear set of goals and the appropriate mechanisms to implement the agreed objectives; or, an incremental or staged solution, starting with an initial «contract» amongst the various interested parties that provides a framework for building up an over-arching organizational structure over a period of time. Of the two the former is obviously the more dynamic solution, but it is resource-intensive and demands a secure constituency as well as a clear vision at the outset of what the aims and objectives of the movement are. The latter option has the advantage that it can start small, build on a range of existing facilities thus allowing an interactive and shared support network to grow «organically» according to the evolving needs and aspirations of the academic community. While these two development patterns will probably produce differences in the character of the prospective organization, their ultimate goal is the same. They are also equally dependent for their successful conclusion on the long-term commitment of participants to cross-border collaboration and the sharing of resources. If the congress decides in favour of this motion it might find the second route to be the more sensible one to take in view of the resources question, and allowing for the need to respect the identity of already existing national interest groups that have emerged in response to local demand. Whichever is the case, such issues need to be addressed at this meeting so as not to miss a golden opportunity for setting in motion an important new phase in the development of construction history as an academic subject.

Exactly how this is to be executed is the business of the congress and its organizers. The most useful thing that could be done in advance of public debate of the issue is to outline a possible scenario for such a development, identifying the principal factors that need to be taken into account. With this in mind I would like to suggest that the domain of any society or association aiming to promote the cause of construction history nationally or internationally should have the following range of activities as a standard agenda (no particular order):

- The raising of awareness of the field/ discipline within the building industry and related educational programmes.
- The identification and definition of subject boundaries and objectives.
- The representation of the field/ discipline as a significant cultural activity within the wider community.
- The coordination of an information exchange network on the topic.

- The promotion of academic research and the dissemination of its findings.
- The collation of archival material on the subject and the securing of its preservation.
- The establishment of a platform for public debate on issues related to the topic.
- The creation of a framework for social discourse amongst members and other interested parties.
- The formation of links with other organizations and bodies concerned with the historic built environment and its conservation.

Fortunately, because construction history and its practitioners have a wide reference base, there is no shortage of experience of good precedents for creating such an organization. An excellent general model in terms of presentational style, range of activities, and the high academic standards it sets would be the Society for the History of Technology (SHOT). Other international bodies like Icomos and Docomomo offer interesting alternative organizational models with different, but related objectives. Although smaller than these the Construction History Society (CHS), founded as a charity in the UK in 1982, but with an increasingly international outlook has the advantage of having a similar operational brief to that which is outlined above, with many of the vehicles to implement such an agenda already in operation. Experience gained from running the latter and other organizations and interest groups that have been formed to promote the cause of construction history nationally, notably the Sociedad Espanola de Historia de la Construccion (SEHC) in Spain, and the Associazone Eduardo(?) Benvenuto (AEB) in Italy, should provide a firm foundation upon which to build a new international network. It is already clear that whatever emerges on the wider front, there will be a need for national branches to look after the particular needs of local constituencies, so some sort of composite, de-centralized structure seems to be the likely outcome for the projected international body.

If that is the case then the congress should turn its mind to starting the process of constructing the framework for collaboration amongst scholars from different countries. It could perhaps begin by concentrating its initial efforts on a selection from the different channels by which it would seek to put its various objectives into effect, those for which there are either already a working prototype, or good models to draw upon. Four such vehicles immediately spring to mind: 1) An academic journal, 2) a web page/newsletter, 3) an annual series of symposia and, 4) a bi-annual summer school.

1. In *Construction History*, the annual journal of the Construction History Society, currently running to Volume no. 17, the international group already has one of its keystones in place. It is the only *international academic refereed journal* in existence devoted to construction history and has a solid reputation. It could easily be expanded to become the mouthpiece of an international community of scholars in the field.
2. *A web page with an electronic newsletter,* run by one of the national associations with good computing facilities is essential right from the beginning. It would focus on polemic and the diffusion of useful topical information (bibliographical updates, relevant exhibitions & events, research in progress, grants etc.). Again an existing facility of one of the national groups could be expanded which, over time , might develop into an international databank for the subject.
3. Likewise, it should neither be too difficult nor too expensive to set up an *annual series* of short, focused *week-end seminars or «colloques»* for between 50 and 100 people, on selected specific themes with invited speakers. Ideally they should concentrate on the comparative analysis of themes common to different societies and aim for the highest possible level of academic debate, with the edited results published and distributed. The venues for these could rotate and sponsorship should be sought for individual events so as to keep the costs down in and facilitate wide attendance. Already existing construction history groups or educational institutes could act as hosts, or one might base them at international conference centres like the famous «Monte Verita» complex, near Lugano on the Italian-Swiss border. These events could become a sort of flexible «think tank» mapping out the territory for the subject. An impressive model exists in the annual «collogues» organized by the Centre D'Etudes Superieures

de la Renaissance at Tours University, including one of particular relevance to construction historians: «Les Chantiers de la Renaissance» (1983/4). The proceedings were published in 1991 (Guillaume 1991). A notable series of annual colloquiums on the theme, Architecture & Behaviour, is run by the Federal Institute of Technology, Lausanne, Switzerland, published as, *Comportements.*

4. A regular series of *bi-annual summer schools* could be organised on much the same lines as the seminars, and would be complementary to them. One could envisage these to last for about a week and be built round visits and expert analysis of interesting historic structural developments or themes, e.g. railway architecture, dockyards, cathedrals etc.

This incremental process of building the network would not only be reasonably economical to implement; it would also provide the widest possible participation at different levels, from the individual to the institution. In my experience the success of this kind of organization depends very much on the enthusiastic participation of high calibre individuals (supported by their home institutions), and everything possible should be done to ensure that there is maximum opportunity for individual scholars and others with a subject interest to participate creatively. Periodic congresses like the one in Madrid are useful for taking stock and to determine general future policy. Again this could eventually become part of a set pattern to complement the other events, once these have been put on a firm footing. Regarding overall management: an open self-regulating network like the above requires minimum central bureaucracy —a «kitchen-table-style cabinet» plus a postal address, with an international advisory board like the one created for this congress, meeting once a year will probably suffice. The important thing is to keep the collegiate spirit of the academic community alive.

Hentie Louw. Newcastle upon Tyne. 4 November 2002.

Notes

1. Merritt Roe Smith (Smith and Marx 1994: 1-30), for example, discusses this issue in relation to post-Republican American culture.
2. Bertrand Gille's classic, *The Renaissance Engineer* (Gille [1964] 1966) is a rare attempt at exploring a broader technological theme for the era.

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Reflections on the related histories of construction and design

Rowland Mainstone

Construction is always a means to an end: to provide passage over an obstacle, to enclose space and provide shelter, to serve as a containment or barrier. Its history is therefore intimately linked with the broader histories of architecture, bridges and the like, and the histories of individual works.

But if this history is to be recognised as a separate discipline we must take a narrower view.

I shall accordingly concentrate here on two aspects to which, for many years, I have continually returned while pursuing an interest in the broader histories alongside a primary concern with design today. They are the two things that distinguish all human construction from the natural growth of living forms. One is the construction process. During this process the built form is only an incomplete and sometimes hardly recognisable approximation to what is finally intended and may not even be able to stand unaided. The living form is always complete and self sufficient as when, for instance, seedling develops into sapling and sapling into tall tree. The other is human choice. At most this can marginally influence natural growth. But, as exercised in the process of design, it is central to all construction. It determines the final form and the processes whereby it is realised. It may be exercised once for all. More often it is seen as a sequence of choices made as the design is developed and as construction proceeds. Then especially, it is closely related to the construction process and this interrelation will be a secondary theme of what follows.

Tracing the histories

Tracing the histories of both construction and design processes presents the greatest challenge when we look back beyond the 18th century.

As we approach the present day the picture, although more complex, becomes clearer. Far more of what has been built survives and there is an increasingly copious, readily accessible, and easily intelligible contemporary documentation beginning with new types of treatise such as those of Belidor (1729) and Rondelet (1814) and detailed accounts of particular works such as Perronet's bridges (Perronet 1782–83) and the Eddystone lighthouse (Smeaton 1793). There is also plentiful evidence for related developments in materials, in construction plant, in the understanding of structural behaviour and requirements, in the ability to analyse and predict these, and in the powerful aid now offered by computers in such analysis.

Since there is already an almost equally large corpus of relevant studies of its history, it seems unnecessary to do more here than outline some salient features of this more recent period before turning to what happened earlier.

Construction possibilities and the construction process since the 18th century

Construction possibilities are determined in the first place by the available choice of materials. Over a

little more than two centuries that choice has widened greatly and that widening has been increasingly a global phenomenon as a result of easier transport and world-wide trade.

It began with the large-scale production first of iron and then of steel. Both materials called for new kinds of industrialised off-site fabrication of the beams and columns and other components that replaced earlier timber counterparts and for new ways of jointing them on site.

Next came Portland cement followed by the highly versatile composite material, reinforced concrete. Improvements in the quality and strength of the concrete coupled with the development of higher-strength steels then presented the possibility of prestressing the steel and concrete.

More recently there have been numerous further developments including the introduction of steels of much higher strength, aluminium alloys, and various plastics.

New methods of fabrication and erection have also been introduced. They have included the use of sliding formwork for casting concrete, cantilevered launching or extrusion of long spans from a support at one end, the jacking up of elements and even complete floor systems after casting and of timber grids after initial assembly on the ground, and the use of internal air pressure to inflate large membranes. Complementing them have been new techniques for accurate setting out and for controlling the erection process.

Most important among these developments has probably been prestressing, not only of individual components of a structure such as precast beams but of assemblies of components and major parts of complete structures. Since it can alter significantly the way in which the loads will be supported, it confers new freedoms on the choices of both erection procedure and final form.

DESIGN SINCE THE 18TH CENTURY

To exploit all these new possibilities in design, an adequate understanding of them and an ability to quantify requirements and assess whether they will be met have been essential. Reductions in weight that have become possible in spite of large increases in other loads have made it essential to consider these other loads explicitly, and the exploitation of tensile and flexural continuity has called for new understandings of strengths and stiffnesses and the parts they play in ensuring overall strength and stability. Successive developments from earlier largely intuitive understandings of structural behaviour to the more precise quantitative understandings made explicit in modern theory have therefore been highly important parallel changes.

These latter developments were initially an outcome of purely disinterested enquiry into the characteristic strengths of different materials and the balance of forces acting in any direction and not merely vertically.

The resulting simple statical theory made possible, in 1742, the first recorded analyses to assess the

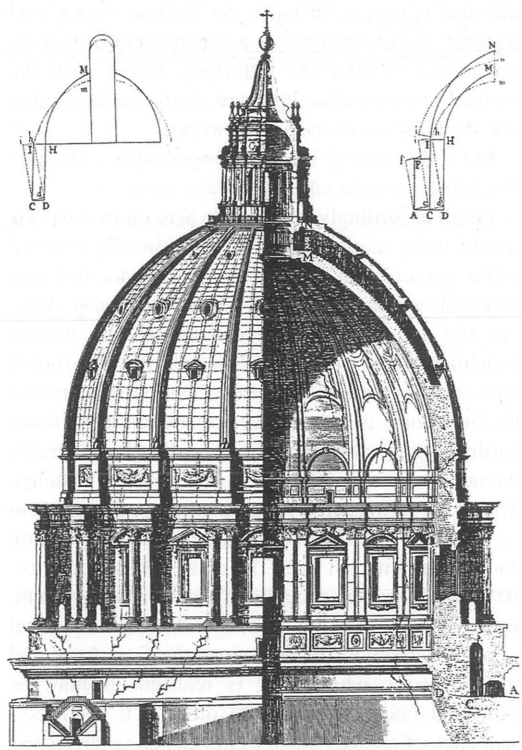

Figure 1
Simplified models of possible collapse modes of the dome of St Peter's based on observed cracking to permit analysis of its stability. (Le Seur, Jacquier and Boscovich)

Figure 2
The Britannia Bridge before major reconstruction after serious damage to the tubes by fire. The piers were carried well above track level to carry suspension chains later dispensed with. (author)

stability of a standing structure, Figure 1. (Le Seur, Jacquier and Boscovich 1743).

Less than two decades later, similar analysis was pressed into service for the first time to justify a highly innovative design of what became the Paris Panthéon while it was still in the earliest stages of construction (Gauthey 1771).

The first extension of this understanding came when other loads became as important as self weight or more important, and when more costly iron replaced timber or masonry and created a new incentive to make the best use of the material. It became common practice to apply proof-loads to columns, beams, and the links of suspension chains and the like, before they left the foundry. Then, to reduce this dependence on tests, it became desirable to be able to predict both strengths and stiffnesses.

Prediction involved taking into account the relationship between load and deformation or, as we should now say, between stress and strain —initially on the basis of what is now known as Hooke's Law. Doing this was not as easy as had been the application of simple statical theory to arches and domes. It was necessary to look afresh at each new form and test the validity of the predictions, the testing also serving to carry understanding further.

The power of the new approach was well demonstrated in the design of the Conway and Britannia railway bridges (Fairbairn 1849; Clark and Stephenson 1850).

Once the idea of making the spans tubular had arisen, exploratory tests were made on small tubes of various cross-sections followed by tests to failure on a much larger model tube which was modified after each test to eliminate the principal weakness that had been disclosed. By the time that the last of these tests had been made, enough had been learnt to proceed to the detailed design of the single-span Conway tubes. Measurements on these after erection provided confirmation of the calculated deflection and assisted in determining the amounts by which the outer ends of the outer Britannia tubes had to be raised above their final bearings before connection to the central spans to make all three spans continuous.

Subsequent developments in theory have greatly extended the possibilities of drawing on past experience

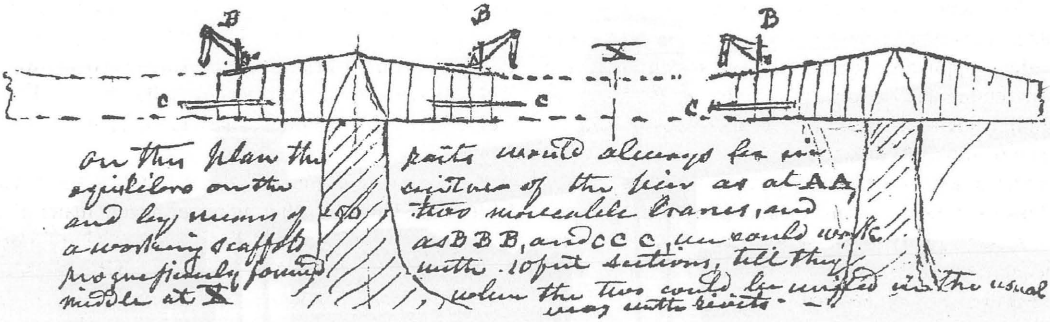

Figure 3
Another early idea, abandoned on account of the likely difficulty of keeping them balanced, for constructing the tubes in situ by cantilevering from the pier heads. (from a letter from Fairbairn to Stephenson now in the archives of the Institution of Civil Engineers)

to foresee and analyse the performance of a projected new structure both during construction and afterwards, and the advent of ever more powerful computers has immeasurably speeded the analysis. This has made it far easier to compare alternative designs and even sometimes to generate them automatically within prescribed limits. Where tests have still been called for to validate new theories or analyses of particular designs, they can now be performed more expeditiously and much more can be learnt from them thanks to developments in instrumentation and the recording and processing of data.

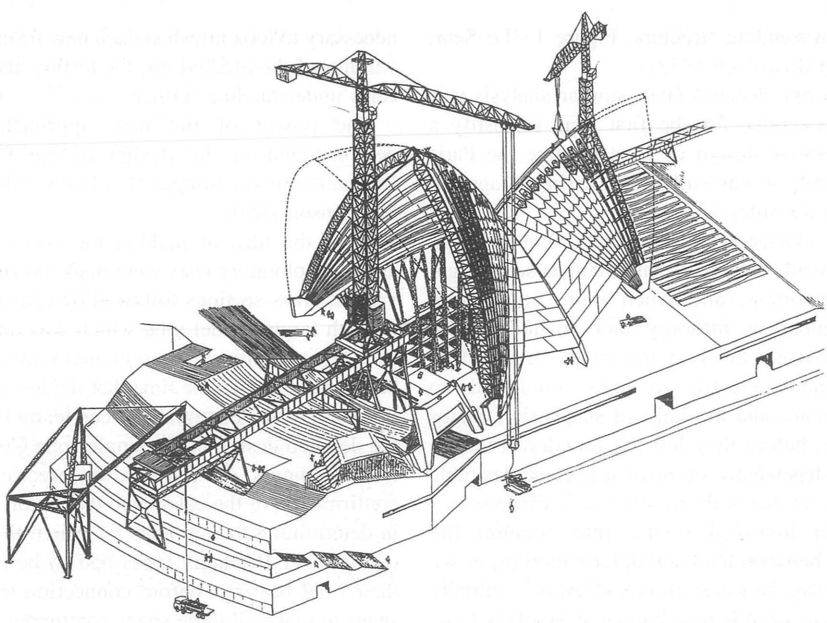

Figure 4
The means devised to erect the ribbed «shells» of the Sydney Opera House. (drawing by Yuzo Mikami)

Complementary studies of likely loadings and other requirements have provided the necessary more widely applicable bases for assessment that are now commonly set down in Codes of Practice and similar advisory or regulatory documents. Cautious specification of limits allows for most inevitable uncertainties although they cannot in the same way allow for human error and unanticipated consequences of venturing into new territory. But when failures do occur, more searching enquiries than were previously possible allow revisions to be made to reduce the likelihood of repetitions.

Through all these developments, a designer's potential personal understanding of the range and limits of structural feasibility has been greatly enlarged, allowing the creative imagination to range more freely and inspiration to be sought more widely in the initial conceptual phases of design. But to attain that understanding calls for considerable effort and experience. Together with the increasing range of other requirements to be satisfied, this has led to the growth of a number of different design professions and to multi-professional working on projects of any size.

An outstandingly successful recent example was the collaboration between Utzon and Arup in the difficult realisation of most of Utzon'a dream for the Sydney Opera House —a realisation in which devising the construction procedure for the «shells» was a more than usually integral part of the design. (Arup and Zunz 1969; Mikami 2001)

Some of the best modern work, including many of Maillart's, Freyssinet's and Leonhardt's bridges and Arup's Durham footbridge, has nevertheless probably resulted when, as for the Conway and Britannia Bridges, the requirements have been largely or purely structural and the designer has been responsible for the construction procedure as well as the structural details and final form.

EARLIER BUILT FABRIC AND FORMS

When we turn to the earlier period, we can only speculate about the beginnings. It is nevertheless reasonable to assume that the earliest wholly built shelters most closely resembled the simplest ones that we still see in many places around the world: round beehive-shaped huts constructed from readily available local materials. The archaeological evidence is at least consistent with this possibility and, as with the typical bird's nest, only the simplest operations would have been called for to build them. There would be no corners to turn and no difficult joints to make.

We are on surer ground when we look at survivals built of masonry and other more permanent materials. Many survive only in part. But their partial collapse or loss of facings has often laid bare details of their structural fabric that would have been hidden in the finished structure. Much attention has already been given to these details and there is a copious record of them beginning with the pioneering studies of Viollet-le-Duc (1860), Choisy (1873 and 1883), Clarke and Engelbach (1930) and Orlandos (1955,1958).

With some exceptions, the materials used were of fairly local origin and the manner of use indicates strong local traditions. The most durable materials were initially natural stones followed by fired brick and Roman concrete. Although strong in compression, these materials had limited tensile capacity and hence were also relatively weak in bending. Where greater tensile or bending capacity was called for, timber was the usual choice, with iron employed to a limited extent and rope and woven fabrics for some types of temporary shelter like the Roman velarium. Problems in effectively jointing these latter materials greatly reduced their effective tensile strengths in many situations however.

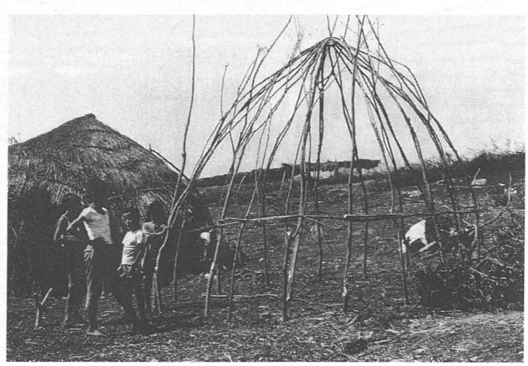

Figure 5
A simple hut under construction in northern Greece in 1958. (author)

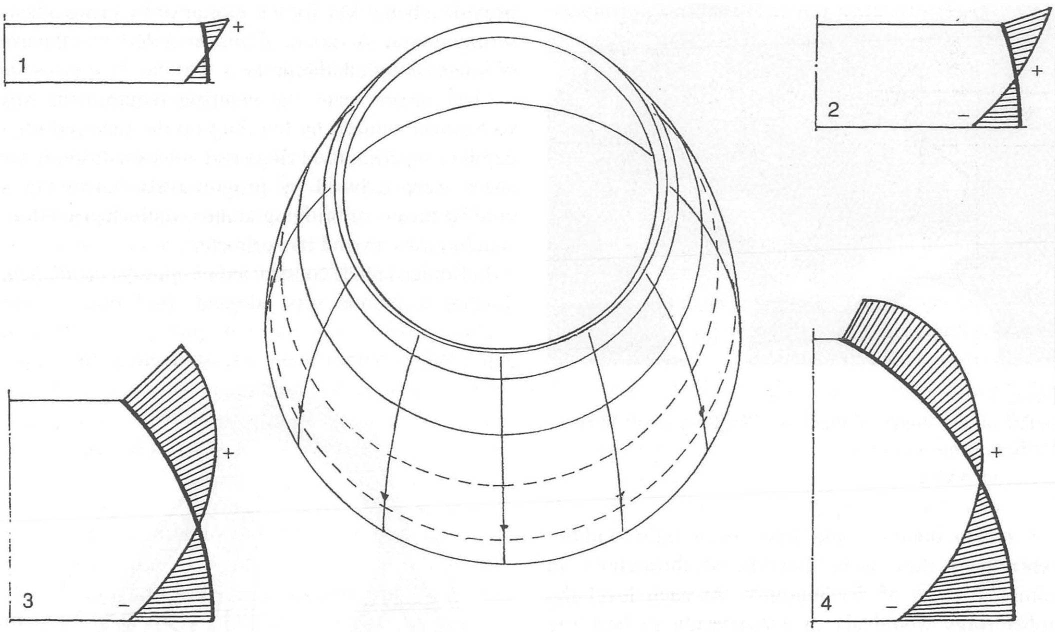

Figure 9
The development of hoop stress as construction proceeds in a circular dome of pointed profile such as that contained within the Florence dome. Tension is shown as negative and compression positive. (author)

inserted at intervals between groups of broken facing bricks to divide the otherwise undifferentiated concrete mass into voussoir-shaped blocks, Figure 6. These would have reduced the risk, particularly in flat or segmental arches, of its partial collapse and thereby allowed earlier removal of the supporting formwork.

In a similar manner, effective groin and meridional ribs were created within the concrete of vaults and domes as shown in Figure 7 and (somewhat idealised) in many of Choisy's plates, though their value in reducing centring needs as distinct from channelling thrusts to supporting piers is less easy to assess.

The outstanding achievement was the construction of the vast double-shelled non-circular dome of Florence cathedral without using centring. Since I have discussed this in detail elsewhere (Mainstone 1969/70; Mainstone 1977a), I now merely outline and illustrate its most significant features. The basic idea was to make the inner shell thick enough to contain within its thickness at all levels an adequate complete circular ring (shown stippled in the larger detail). Above the level at which the inward inclination became appreciable, this whole shell was then constructed of brickwork with inverted-conical setting beds as if it were part of a thicker true circular dome. Construction of successive courses without temporary local support was made possible by means of the ingenious bond illustrated in the details. This also keyed to the circular ring the masonry inside and somewhat below it, and was probably suggested by the Roman construction shown in Figure 6 (Mainstone 1980). Construction was further simplified by the pointed profile and an open eye at the top, later covered by a lantern.

This achievement was also uniquely well documented, notably in a series of agreed specifications of the design (Doren 1898). Interpretation of these specifications has nevertheless presented difficulties in the past, partly because existing terms had to be pressed into service to describe new procedures and forms. To understand them fully it was necessary to read them

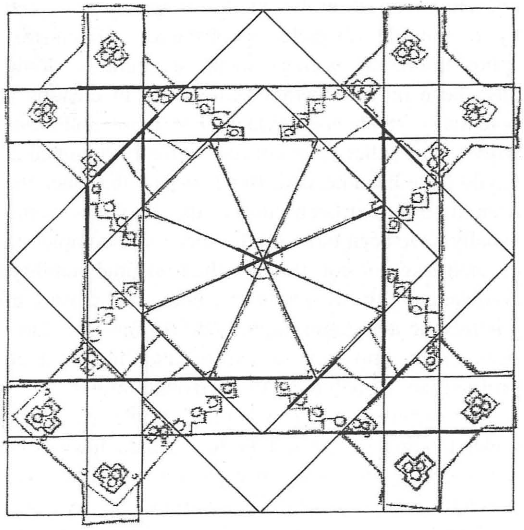

Figure 10
Plan of a tower of Laon Cathedral from Villard de Honnecourt's «notebook» (author, based on f. 9v of *Bibliothèque Nationale MS français 19093* with the addition of original scribed construction lines visible in glancing light)

with a full appreciation of the structural possibilities and needs and the consequences of proceeding in different ways.

To confirm this understanding, a rough idea of the development of hoop stress as construction proceeded was obtained by analysing a simple model of the circular dome contained within the thickness, Figure 9.

Earlier design

Modern structural analysis is, however, of no direct value in establishing earlier design processes. Indeed much finite element analysis of early masonry structures now undertaken is grossly misleading even as a basis for assessing present stability because it ignores the influences on behaviour of the heterogeneity, extensive cracking, construction breaks, and sequence of construction. On the other hand, analysis that takes proper account of these can assist by clarifying the successive structural requirements of different construction procedures as in the Florentine study. It can also throw some light on problems that might arise and on the ways in which they would have become apparent to the builders, and can contribute to a better understanding of the significance of the visible evidence presented by a standing structure (Mainstone 1997). In attempting to envisage how our predecessors might have proceeded, we must merely divest ourselves of most other insights such analysis offers and try to think as they could have thought.

Contemporary documentation is also of limited value. What was learnt directly on the worksite seems largely to have gone unrecorded. This poses a risk of placing undue evidence on what was recorded and has come down to us. Since, also, the records were not intended for our enlightenment, they are today even less self-explanatory than the Florentine specifications.

Building regulations did little more than place restrictions on what was permitted and were mostly

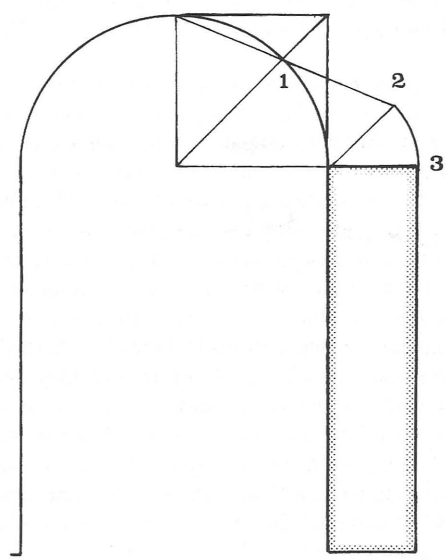

Figure 11
Geometric constructions for establishing pier widths required to buttress arch thrusts. (author, based on drawings in S. Garcia, *Compendio de arquitectura*, c.1681, and E. Derand, *L'architecture des voûtes ou l'art des traits et coupe des voûtes*, Paris, 1643)

concerned with a limited range of building types and with aspects like fire safety. Other surviving records from Vitruvius onwards are largely restricted to simple proportional rules and simple geometric procedures that could have served equally for design, setting out on site, or stone cutting (Mainstone 1968). Even where they do relate specifically to such things as room proportions on the one hand or wall thicknesses and column girths on the other, it is mostly unclear whether they were intended to contribute to firmness or convenience and delight.

The proportional rules for wall thicknesses and column girths and the like and later geometric constructions to determine buttressing requirements, Figure 10, would simply have been codifications of past experience that certain proportions had proved adequate. Close conformity to them, or more directly to the experience itself, must usually have served as the best available assurance of safety when something new was attempted. Indeed a failure to conform more closely to the proportions previously adopted for the piers at St Peter's, St Paul's, the Val de Grace, the Invalides and elsewhere was still the main basis of Patte's criticism of Soufflot's original design for rebuilding the church of Sainte Geneviève (Patte 1770).

The purposes served by generalised procedures based on manipulating a few geometric figures as in Figure 11 are more enigmatic. How far were they thought to be valid guides to more innovative structural design? The minutes of the late 14th century discussions of the proposed design for the nave of Milan Cathedral suggest that, at least in part, they were (Ackerman 1949). But they contained in themselves no clues as to how they should be interpreted and choices made between alternative possibilities. It must be assumed that they were supplemented in practice by unwritten further rules as the discussions at Milan also suggest. Even so, they must have been flexible enough to permit the large variations to be seen from building to building where it is to be expected that the same procedures would have been followed.

Clearly, successful innovative design must have had some further basis (Mainstone 1973), though it could have been much simpler than today because of the simpler modes of behaviour brought into play. For the earliest huts it was probably no more than an appreciation of the natural tendency of everything to fall and a realisation that the fall might be prevented by a suitable obstacle or obstacles. Given this appreciation, the primary focus of attention would have been on the operations involved in creating a potentially stable assemblage. For more ambitious works some fuller prior concept of the form aimed at would have been needed. But I suspect that once the desired form had been chosen, the emphasis would usually have been more on the successful completion of each stage of construction than on final stability. Avoidance of the difficulty of a horizontal closure of a hemispherical dome might well, for instance, have been a more important reason for adopting a pointed profile than the reduction of horizontal thrusts.

As experience grew, the simple intuitive understanding of the hut builder would have been developed by the experience gained in tackling any problems, coupled with observation of the evidence of their behaviour presented by standing structures. Some further guidance may occasionally have been sought by constructing reduced scale models as happened in the early stages of design of the dome in Florence. But there is little evidence of this happening elsewhere. Even in Florence, the last clause of the initial design specification for the dome still left open the procedure to be adopted in the later stages of the work because «in building, practice teaches what should be done», clearly indicating the importance placed on critical observation of the actual progress of construction as the best guide to what should be done next.

The importance of observing behaviour as construction proceeded was also well demonstrated by the construction history of another major achievement, Justinian's Hagia Sophia in present-day Istanbul (Mainstone 1988). Study of the building today has shown that, when construction of the main transverse arches generated significant thrusts, the original interconnections of the piers lettered A,A began to give way. This allowed the piers to tilt alarmingly, especially at gallery level, and it was only then that the projections lettered B,B, constructed entirely of closely fitting stone blocks, were added in hasty response, Figure 12.

Because design as innovative as seen at Hagia Sophia and in Florence was rare, there would have usually been less need for a similarly empirical approach. The development of the Gothic structural system, for instance, went ahead by numerous much

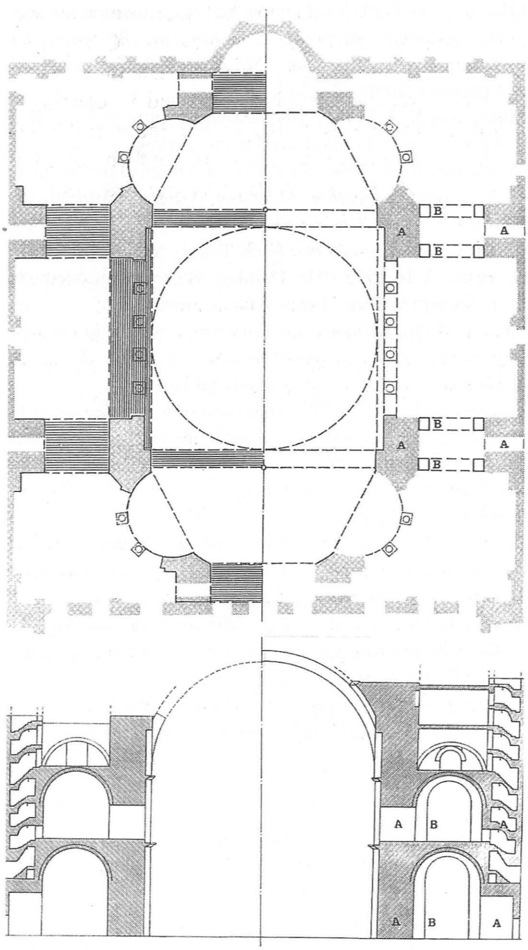

Figure 12
Design changes during construction at Hagia Sophia. The original piers and the arches and main vaulting system they were clearly intended to carry are shown in plan and transverse cross section at the left, and additions to the piers and their interconnections about half-way through construction at the right. (author)

smaller advances. But the need would not have disappeared. Difficulty would have continued to be experienced in choosing cross sections and all else that would determine strengths and deformations under load —including speed of construction when these characteristics would have been time-dependent as they were in Hagia Sophia— and alarming movements were then similarly countered by strengthening weak elements or adding restraints such as buttresses or ties. Since the behaviour observed would have been effectively statically determinate, no more was needed.

Continuity and change

Looking back over the whole history (Mainstone [1975] 1998), I see both continuity and change. Changes between earlier and more recent times have been emphasised. But it should not be assumed that there was a complete change at some time in the 18th century. Major changes began then. They continue at an increasing pace. But they have not affected all construction equally and perhaps never will. Nor have they affected every aspect of design. The behaviour of the incomplete structure at all stages of construction must still be borne in mind even when focusing primarily on final behaviour. So there is always a link with the construction process. Except when no more is done than play variations on what has been done before, keeping well below the ceiling of proven practice, creative input is required. This still calls for personal understanding of all that is relevant and for skilled professional judgement, and runs risks that perhaps can never be wholly eliminated.

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Bridge technology and historical scholarship

Tom F. Peters

Bridges have always fascinated both the layman and the professional. Their structural concepts are simple and visually easily understood: they are linear, carry traffic, and cross a gulf. Symbolically their concept is more complex: they span from one realm to another, cross the deep uncertainties of «troubled waters,» and connect. Bridges demonstrate human ingenuity and the triumph over nature, contradict the physical limitations of gravity by levitating traffic in the air, and make the impossible reality. Cultural historians and theoreticians love them, and it is not by chance that the Pope carries the title of *pontifex maximus*, the «supreme bridge-builder.»

Because of their linear simplicity and structural clarity, bridges also provide ideal case studies for that down-to-earth group that we represent, the historian of technology. They can teach us how diverse technological thinking is and how our viewpoint of technology changes over time and even how it varies between the fields of engineering, architecture, and construction.

Architects and engineers view the same thing from entirely different standpoints. For istance, a connection in the Bayonne Arch Bridge built in New Jersey by the engineer Othmar Amman in 1931 gives differing information to engineers and architects (Figures 1 & 2). When asked what they see in the

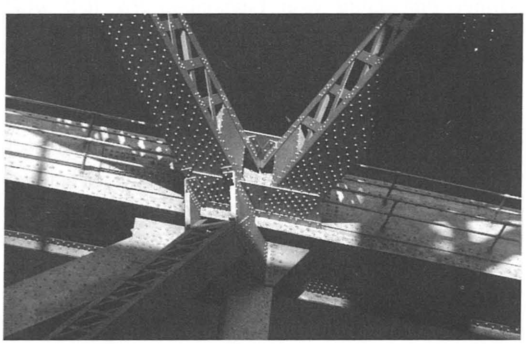


Figure 1
Steel connection on the Bayonne Bridge over the Kill van Kull, New Jersey by Othmar Ammann, 1931 (photo: T. F. Peters)

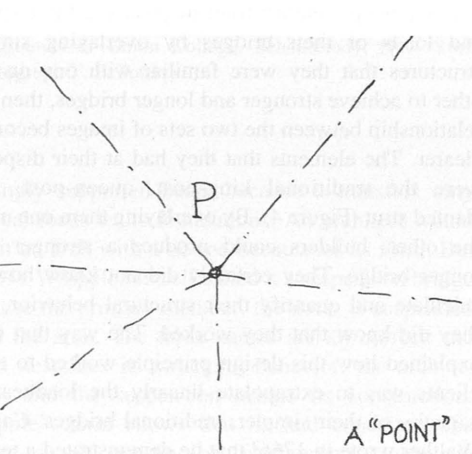


Figure 2
The diagram of a «point» that the engineer sees in the connection (diagram: T. F. Peters)

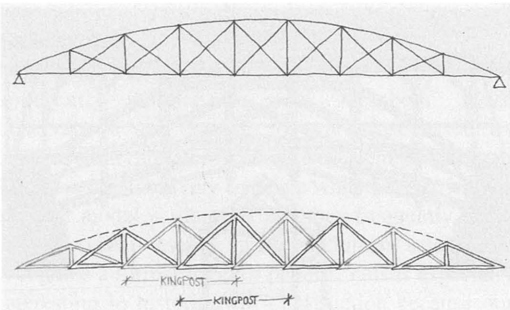


Figure 7
King-post interpretation of Whipple's Bridge (diagram T. F. Peters)

construction, but at its geometrical configuration. Culmann had been educated in the French analytical engineering tradition, initiated in 1797 at the Ecole polytechnique in Paris and developed in the younger German engineering school he attended in Karlsruhe. The German system based its curriculum on a change the Ecole centrale had introduced in 1829. The first tradition initiated analytical thinking in building and the second introduced a practical, industrial component into design.

On the background of this development, Culmann shifted his viewpoint half a module from Whipple's traditional overlay approach and saw what we now call panels. The diagram taken from his publication of his voyage in the Allgemeine Bauzeitung of Vienna proves it (Figures 8 & 9).[3] This is only a slight shift in

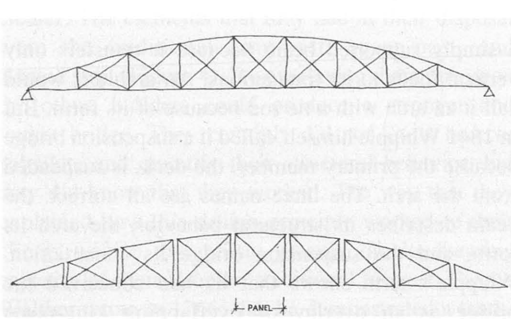


Figure 8
Panel interpretation of Whipple's Bridge (diagram T. F. Peters)

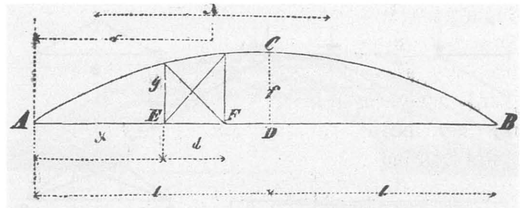


Figure 9
Illustration from Carl Culmann's article of 1851 showing the Whipple Bridge. Allgemeine Bauzeitung, Vienna

viewpoint, but a profound conceptual shift. It changed nothing physical in the bridge itself, but it changed how we see bridge building forever.

We find such shifts at key moments of change throughout building history. When Giovanni Poleni examined the cracking of the dome of Saint Peter's Basilica in Rome in the mid-eighteenth century,[4] he discovered that the line of thrust in an arch or dome was nothing other than the catenary, the line a suspended chain follows, turned upside down

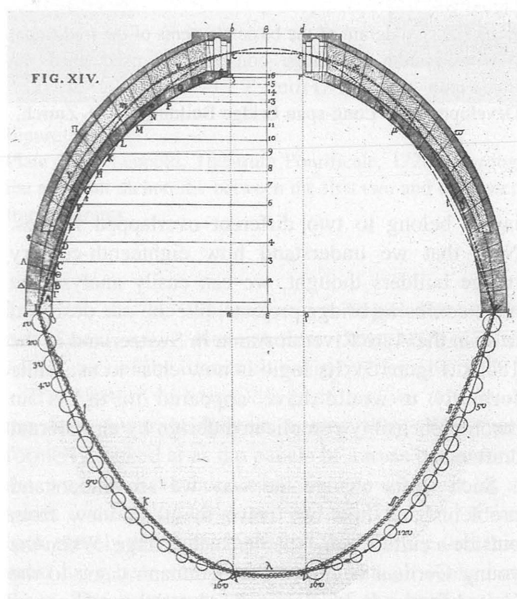


Figure 10
Giovanni Poleni's diagram of the inverted catenary as the line of thrust in a dome, from Memorie istoriche, 1748

(Figure 10). In other words, by inverting gravity he converted a suspension structure into an arch. Poleni's discovery used physics as a tool, and it initiated «hard» analytical thinking into building. Inversion is one of the methods that can shift our viewpoint.

Robert Maillart experienced a similar shift in his viewpoint when his first independent structure, the small Zuoz Bridge over the Inn River in Switzerland twisted slightly and cracked its spandrels in 1901 (Figure 11). Any «normal» engineer would have been horrified and determined to strengthen the spandrels next time so that the failure would not occur again. But Maillart shifted his viewpoint and did the opposite. Since the bridge had not collapsed, he argued, the cracked spandrels must be redundant because they were still holding even though they were not carrying any load. So when he designed his next bridge, the Tavanasa Bridge not far from the first one in 1905 (Figure 12), he left them out and created a distinctive new bridge type. This type of shift never stops. Christian Menn did it again when he inverted the Tavanasa Bridge in 1980 and reinterpreted it as his prize-winning Ganter Bridge in the western Swiss Alps (Figure 13).

Our Western culture is not the only one to use shifts in understanding to innovate in bridge building. In 11th century Song-Dynasty China, a novel bridge type suddenly appeared that has no corresponding version anywhere else. The so-called «rainbow bridge» (Figure 14) has long been the object of

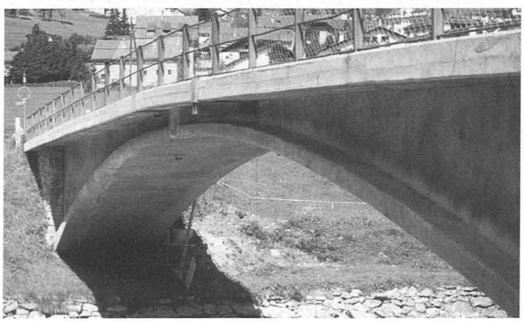


Figure 11
Zuoz Bridge by Robert Maillart over the Inn River in the Engadine, Switzerland, 1901 (photograph T. F. Peters)

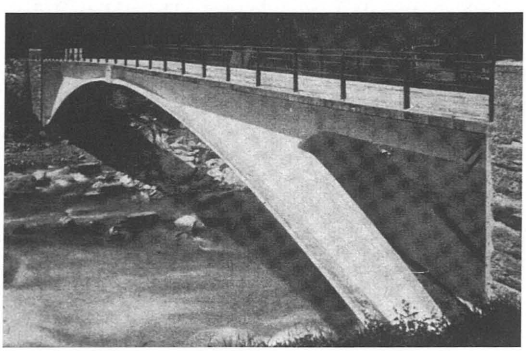


Figure 12
Tavanasa Bridge by Robert Maillart over the Vorderrhein, Switzerland, 1905, from Versuche und Erfahrungen an ausgeführten Eisenbeton-Bauwerken in der Schweiz 1924–1937, EMPA Zürich, Bericht 99 (1937)

Figure 13
Ganter Bridge by Christian Menn on the Simplon Pass Road, 1981 (photograph C. Menn)

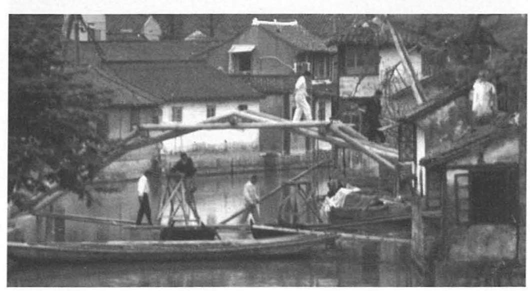


Figure 14
Reconstruction of an eleventh-century «Rainbow Bridge» in Jinze near Shanghai in 1999 for NOVA «China Bridge», Public Television, Boston (photograph T. F. Peters)

speculation in the history of Chinese bridges. Like the overlaid structures of the West, it does not behave according to modern statics, but depends on the pinching action of its woven form for its integrity It seems to be an arch, but the strange thing, as the reconstruction of one in 1999 showed, loading it produces no thrust at the abutments. So it must be a simply supported beam, an engineer would say. But it has no tie rod like the Whipple bridge; so how does it work? Although we are not yet quite certain, it seems that the pinching action of the scissor-shaped connection (Figure 15), holds the crossbeams tightly and somehow takes up the forces that would otherwise flow along the arch-shape to the abutments. This may be analogous to the behavior of the intricate basket weaving that the ethnic group is known for that lives where this type of bridge first appeared. This explanation is, of course very much more speculative than the more easily proven examples from the West, but it is also almost 1000 years older and contemporary documents have long since disappeared.

There is one Western example that also uses pinching as a connection, and that is the first bridge over the Rhine that Julius Caesar is said to have ordered built for his conquest of the north (Figure 16). The Caesar bridge, of which we have only a reconstruction from the description, was a trestle construction with wedges set between the beams that carried the decking and the slanted posts that supported them. These wedges were so arranged that they bit ever tighter into the wood and pinched the beams as the deck was loaded. The solution was conceptually similar, and yet not as simple as the Song Rainbow Bridge, in which the deck beams themselves formed the wedges.

We can learn a great deal from bridges. By following the subtle clues they give us, we can gain insight into how builders used to think and how that thinking was similar to or different from ours. It is important for us as historians and educators to understand that technologists, all depending on their field, their education, their culture, and the period in which they live, can see and think in many different ways. It puts our own time, our own culture, and our own systems of thought into perspective, and that can, I hope, make us more receptive to innovative ways of solving our problems today.

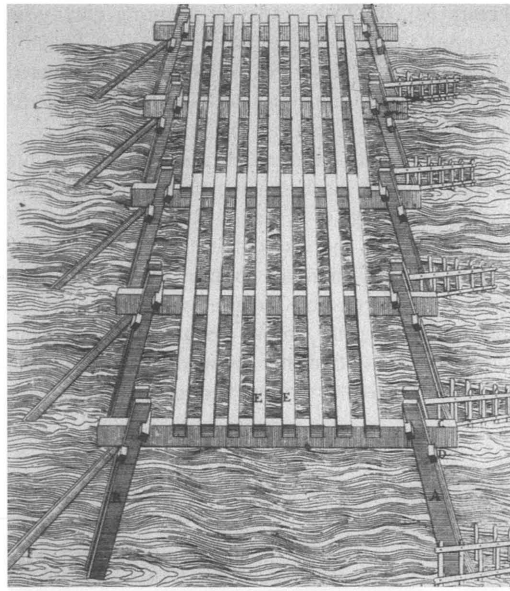


Figure 16
Conjectural reconstruction of the trestle bridge ordered by Julius Caesar over the Rhine showing the pinching action of the wedges between the supports and the deck beams, from the Isaac Ware English translation of Andrea Palladio's Quattro Libri, 1738

Figure 15
Detail of the reconstruction of the «Rainbow Bridge» in Jinze showing the pinching action of the scissor-shaped connection between the arch members and the deck beams (photograph T. F. Peters)

Notes

1. Jacob Leupold, Theatrum Pontificale oder Schauplatz der Brücken und Brückenbaues [sic] . . . 1726, Leipzig
2. Caspar Walhter, Brücken-Bau oder Anweisung, wie allerley Arten von Brücken, sowohl von Holz wie von Steinen, nach den besten Regeln der Zimmermannkunst dauerhaft anzulegen sind . . . 1766 Augsburg.
3. Carl Culmann, «Der Bau der eisernen Brücken in England und Amerika». Allgemeine Bauzeitung, Vienna.
4. Giovanni Poleni, Memorie istoriche della gran cupola del Tempio Vaticano. 1748, Padova.

Stereotomy, a multifaceted technique

Joël Sakarovitch

The meaning of the word stereotomy, the etymology of which broadly designates the art of cutting three-dimensional solids into shapes to be assembled, is restricted in architecture to designate more specifically the art of stone carving for the purpose of constructing vaults, squinches, cupolas or flights of stairs . . . Although universal dictionaries mention «wood stereotomy» as involving the assembly of timber pieces, it is noteworthy that this meaning generally disappears from architecture dictionaries. The shift in meaning is of course not fortuitous and this will be discussed further below. *Vocabulaire de l'Architecture,*[1] defines stereotomy as being «l'art de tracer les formes à donner aux pierres (et aux briques) en vue de leur assemblage», in other words «the art of drawing the shapes to be given to stones (and bricks) for future assembly». Hence it adopts once more the definition given in Aviler's dictionary of 1691, which was considered the authority in the 18th century, and echoes the French expression «art du trait» or art of line drawing. Thus reduced to the «art of line drawing», stereotomy would appear to be solely concerned with the art of drawing lines in preparation for the future assembly of carved stones. In that sense, stereotomy would not be as such a construction technique but merely a preliminary step in stone vault construction. I cast my preference here on a definition attributed to Claude Perrault, a fine expert on the subject, according to whom stereotomy is «the art of using the weight of stone against itself so as to hold it up thanks to the very weight that pulls it down».[2] Being a broader definition than the one in which it is seen merely as the art of line drawing, it seems to me to be more in accordance with the majority of contributions that have been classified in the section on stereotomy in this conference. This implies viewing stereotomy as part and parcel of the construction technique itself, and specifically in the domain of stone construction. I shall therefore talk only about what is directly relevant to stereotomy as such —insofar as the distinction can be made— and not about stone carving in general, the material itself, the different qualities of stone, the various tools, etc.[3] Furthermore, the fact that Perrault's definition identifies the arch as the origin of stereotomy gives a historical depth of 23 or 24 centuries, which is quite an advantage for the historian.

Fundamentally, the problem of constructing a stone vault or any building with stone arches is first and foremost one of statics. The major objective is to resolve a problem of spanning or covering. Now Perrault's definition is indeed formulated in terms of vault mechanics. At the same time, the shape of voussoirs is essential to this art. The underlying geometry is subservient to vault statics and is therefore constructional in the full sense of the word. From this fundamental, intrinsic and consubstantial overlap emerges a polymorphy of stereotomy, a multitude of possible approaches, a variety of ways in which it can be viewed, studied and therefore historicised.

Rather than presenting a panorama of more or less recent research on the subject,[4] I have chosen to try

and present the different approaches proposed by the researchers who decided to climb this great mountain —which is noteworthy for both its height and antiquity (a rare occurrence in geology). My intention is also to analyse the relationship between different approaches and the perspectives they offer and reveal. Cross-examination allows one to show the importance of stereotomy in the history of construction and I think it can contribute to defining the specificity of the history of construction.

Stereotomy can be studied from the standpoint of its relationship to the history of architecture, the applied geometry used by the stone cutter, the erudite geometry of the mathematician, studies in the field of mechanics, and the history of crafts and their emergence.

STEREOTOMY, ARCHITECTURE AND CONSTRUCTION

As a construction technique, stereotomy allows the creation of architectural forms. A taxonomy of forms and their evolution in time and space, as well as a comparative study of national characteristics have produced some of the most beautiful studies on the subject. The use of stone vault construction over a long historical period opens a field —one might even say an ocean— of possible research avenues. Indeed from the emergence of stone vault construction and its spread in Antiquity right through to its golden age and ultimate decline, there have been noteworthy topics available for study. The more readily identifiable are: the birth of complex stereotomy in paleochristian Syria, the development of European stereotomy after the return of crusaders, the comparison between Roman and Gothic stereotomy and how they answered different needs and demands[5] . . . Half historian of construction and half historian of architecture, the researcher must follow the evolution of stereotomy in space and time, with all the difficulties that entails including the precise dating of constructions. Naturally, the evolution in the construction of particular shapes and vaults is linked to the development in the techniques of stone cutting such as repointing, squaring, half squaring, or cutting with a template. Which method was used and when? Which practical and theoretical tools were used and with what result in mind? Etc. And of course the motors for improvement or the reasons for a loss in know-how are also of interest for historians of other construction techniques.

A great number of studies are actually case studies: studies of particular buildings, specific vaulted constructions, or buildings designed by particular architects. Philibert de l'Orme, for instance, remains one of the architects who has been most written about. Even though stereotomy is not the only reason for this, it is nevertheless responsible for a good part of the interest the community of historians has shown in Henry II's architect. The numerous studies dedicated solely to the squinch of the castle at Anet would suffice to show the different possible approaches to vaulted construction.[6] It is true that the destruction of this masterpiece of stereotomy during the French Revolution has contributed somewhat to elevating it to the rank of architectural myth and this squinch is now to stereotomy what Mies van der Rohe's pavillion in Barcelona is to Modern architecture.

Compared to other more recent construction techniques, such as the use of iron and concrete, which were international from the onset, stereotomy has long remained strongly marked by specifically regional and national characteristics.[7] The work of Pérouse de Montclos provides a remarkable repertoire of stereotomic buildings, principally in France but also in the rest of Europe. By presenting stereotomy as the touchstone of French-styled architecture, Pérouse de Montclos proves —if that were at all necessary— that the history of architecture cannot be conceived independently of the history of construction.[8]

These studies usually sing the praise of stereotomy. However, in relation to the history of construction in general, one cannot glide over the fact that architects overconfident in the novel possibilities offered by a new technique or too focused on its (immense) possibilities, could sometimes forget or neglect other parts of a building. As Jean-Louis Taupin put it, they occasionally succumbed to the «the rapture of stereotomy».[9]

STEREOTOMY AND GEOMETRY

One of the more specific aspects of stereotomy is the fact that it is a technique that is deeply rooted in geometry. Unlike the carpenter who makes the

skeleton of a particular volume or the ironsmith who determines the envelope, the stone cutter works directly into the mass of the material, which can be given any shape. Concretely, materially, the stone cutter has in front of him a solid piece of three-dimensional space. It follows that stereotomy involves varied surfaces (usually ruled or revolution surfaces) as well as surface intersections.

Because of this complexity, stereotomy generates situations where a preliminary drawing is indispensable. This situation is neither very frequent nor very old. The history of architectural drawing shows that, up to the Renaissance at any rate, the tendency was for builders to avoid preliminary drawings before starting construction since drawings were made only when deemed absolutely necessary. It is more thanks to the stone cutters than the architects themselves that geometral representation was literally «constructed». This occurred by a slow back and forth process between different cutting techniques, which were long used in parallel and in time, constituted a base of «pre-geometric experience».[10] It is from such experience that stone cutting treatises emerged in the first instance and descriptive geometry later on. In addition, stereotomy —unlike masonry— requires a precision of execution, which further pushes the tendency towards making a preliminary drawing.

Since the preliminary drawing is the crux of the 2D/3D transformation, of the conversion of a two-dimensional explicative drawing into a three-dimensional construct, stereotomy is the starting point of the fully-fledged construction site drawing. And the problems linked to projection and the changing of co-ordinates imply very much more subtle geometric reasoning than those involving planar geometry.[11]

It is for this reason that stereotomy is linked to both applied geometry —practised by building guilds— as well as erudite geometry, which is the domain of mathematicians.

Stereotomy, applied geometry and stone cutting treatises

The elaboration of an applied geometry, which lies at the heart of the transformation from planar to three-dimensional geometry, turns out to be sufficiently complex to have sustained the «secrecy» of the stone cutting guild for centuries. This delayed the formulation of the underlying geometrical theory until relatively recent times in comparison with the progress made in other branches of mathematics. To understand how this step was resolved is therefore the focus of numerous studies referring to stereotomy, particularly during the Middle Ages.[12] The stone cutter statutes —promulgated in Ratisbona in 1459— forbade the disclosure of the «guild's ways and practices», which certainly included the way to «draw» the elevation from the plan. These statutes are therefore an integral part of the history of stereotomy in the Middle Ages. Such geometrical knowledge, wrapped in a halo of secrecy, contributed to the «secret of cathedral builders», which, like the secret of the pyramids or the secret of Roman concrete, has always stimulated the curiosity of scholars and potential readers. It is not impossible in my view that, partly for this reason, the so-called secret surrounding such issues has been somewhat exaggerated in a good number of commentaries.

In any case, it is striking to note that the «mystery» surrounding the working drawings of fitters and carpenters is to be found again when descriptive geometry was created. According to Dupin, Monge supposedly declared, when he was teaching at the Louvre in the 1780's, that «Everything I achieve with calculations, I could also achieve with a ruler and compass, but I may not reveal such secrets».[13] Théodore Olivier also recounts an anecdote according to which a civil engineer had had his notes from Monge's course stolen by artillery officers. It turns out the thieves failed miserably in their attempt to «decipher the hieroglyphics» of the Mézières School.[14] The fact that an atmosphere of mystery still hovered when descriptive geometry was being invented is proof of the real difficulty involved in reading and interpreting the drawings that were the key to the 2D/3D transformation. It is indeed a language —and Monge does define descriptive geometry in such terms— a language that needs to be learned. Thus the reference to hieroglyphics is hardly fortuitous but Champollion's talent is not given to everyone.

The relationship between stereotomy and applied geometry also explains the large number of stone cutting treatises written right through into the 19th century, either edited or in manuscript form. Thus the

analysis of original treatises and their reedited copies with commentaries, as well as edited manuscripts are, together with the study of stone vault constructions, an invaluable tool in the study of stereotomy.[15] There again the approach may be architectural, through a comparison of the guilds presented in the various treatises. It can be geometric, through the study of the graphic methods presented and the analysis of the applied geometry used, which may (or may not) brings solutions yet does not provide answers to fundamental questions.

Finally, one ought to mention that, given the quasi desert of sources on technical drawing during the Roman and Gothic architectural periods, Villard de Honnecourt's *Carnet* seems like an oasis of untold riches. The two drawings about stereotomy in this *Carnet* are truly precious corner stones enabling one to appreciate the evolution of graphic methods applied to stone cutting.[16]

Because they are basically dealing with the problem of transposing 2D into 3D, stereotomy treatises also offer one of the most complete examples of the evolution of representational modes in space. For this reason, stone cutting drawings have been one of the motors of the evolution of space representation techniques.

Stereotomy and erudite geometry

While stereotomy, together with carpentry, provides one of the richest examples of the uses of applied geometry, it is also at the root of a branch of erudite geometry, namely descriptive geometry. To sum up the situation, one might say that stereotomy is to descriptive geometry what perspective is to projective geometry.

The parallel between the evolution of stereotomy and perspective is indeed striking. Both practices developed during the Gothic period —whether on stone cutting work sites or in painters' workshops. The first treatises were edited during the Renaissance and the mathematicians of the «Monge School» explicitly theorised stereotomy and perspective at the end of the 18th and beginning of the 19th century. In a letter addressed to the minister of war, the director of the Ecole du Génie de Mézières writes that Monge «has demonstrated the theory of stone cutting»,[17] an expression which successfully expresses where the matrix of the Monge theory lies. Numerous studies, past and present, have focused on the deep, old and complex ties that exist between stereotomy and descriptive geometry.[18] It is worth noting the specific role played by «squaring» in the emergence of a type of geometric thinking that was to generate descriptive geometry. Stone cutting by squaring, which does not have its equivalent in carpentry, has the advantage over the template method of providing an algorithmic process of form discovery. This explains why this method, though more time-consuming and more expensive, has never been totally abandoned in practice. Just like graphical techniques used in cutting with templates, descriptive geometry theorises this algorithmic procedure as well as the definition of surfaces associated with it.

Showing that descriptive geometry was in fact born of the heaviest of all the techniques it more or less theorised, namely stereotomy, weighs it down forever. «Let Descartes intervene, then Monge and many others, they still work as always from the applied as well as the representational standpoint, perpetuating the cleverness of engineers, inducing the survival of archaic, pre-mathematical practice and thus blocking the emergence of science in all its purity. And this science is born precisely when this cleverness dies: not very long ago».[19] Thus Michel Serres makes of Monge the last «harpedonapt». Yet, in the wake of Chasles, no science historian describing the origins of modern geometry would refuse to see Monge as Poncelet's teacher. None would refuse to find in *Leçons de géométrie descriptive* the starting point of a rebirth in geometric studies at the beginning of the 19th century and the beginning of the ensuing profound upheaval in mathematics. Thus, in spite of Michel Serres' assertion, geometry «in all its purity» —that is to say freed of Euclidean metric— was launched in a drawing course for engineers and it would seem that science in all its purity was born of this cleverness.

The German mathematician Felix Klein who claimed «to have been educated . . . thanks to [his] teacher Plücker in the Monge tradition», considered the «application of geometric intuition to analysis» to be one of the major contributions of this tradition.[20] In the *Erlangen Programme*, which is considered to be the foundation of modern mathematics, Felix Klein explicitly refers to this tradition. This is not to suggest that modern mathematics are a direct result of the

spiral staircase of Saint-Gilles. But it means that descriptive geometry, which belongs to the history of techniques through its origins and that of mathematics through its development, establishes a link between the stone cutting tradition and the history of science. And this does confer to stereotomy a rather original position.

Stereotomy and vault mechanics

Vitruve believed that geometry provided simpler rules than did statics for the construction of arches. And until Galileo, it was thought that «geometry —and not mechanics— [was] the true guardian of stability».[21] The «Firmitas» thus mainly belongs to the field of geometry. The new Galilean line of thought imposed itself rather slowly and one cannot fail but notice the total absence of knowledge Guarini, Blondel and Fontana, for instance, had of statics and material resistance. Before the 18th century, builders only had extremely simple, purely geometric and (at best) empirical «rules» at their disposal to size the buildings under construction. One of the most famous rules is the «Leonardo rule», which says the arch will not break if the chord of the outer arc does not touch the inner arc.[22] Another is «Derand's rule», which gives the sizing for the piers of a vault, the size of which is by the way independent of their height. In spite of this construction aberration, the Derand rule —like the Leonardo rule— was still extolled throughout the 17th century and to a large extent during the 18th century.

This state of mind explains why stereotomy has come to be perceived in some ways as what I would call «twice over geometrical». Because, in addition to having a situation objectively requiring an extensive knowledge of geometry —as mentioned above, problems of statics are approached from a geometrical standpoint. This is why stereotomy treatises essentially deem themselves to be books on applied geometry.

Towards the end of the 17th century, the problem of arches and vaults was approached by the European erudite world from a mechanical standpoint. Following this evolution, numerous studies focused on the slow evolution of mentalities on the subject, the difficult switch of thinking in mechanical rather geometrical terms. In other words, they focused on what Eduardo Benvenuto describes exquisitely as the study of how vaults, for which we previously only had solutions, are going to become a problem.

What shape should an arch be? How wide, how thick, how high should the piers of a vault be? One of the earliest answers to this mechanical approach of the problem is Philippe de la Hire's memoir. It deals with the application of lever theory to vault mechanics and has been one of the most extensively studied. The analysis of arch and vault statics is important because it is one of the first examples where infinitesimal calculus was used for practical purposes. The demonstration of Catenary properties by David Gregory or Jacob Bernoulli, Coulomb's memoir on the method of maximis and minimis, the taking into account of material resistance and friction at the end of the 18th century and of the theory of elasticity during the 19th century, constitute an entire chapter in the history of construction and the evolution of thinking in terms of mechanics with respect to the rest of knowledge.[23]

The succession of treatises and memoirs on the subject allows one to study the to-and-fro between the idealisation necessary for mathematisation and the appraisal of the full complexity of phenomena. For instance, how do we go from an infinitely thin to a thick and heavy vault? Since these studies belong to applied or «mixed» mathematics —according to the 18th century French expression— the problem arises as to how to propagate them both amongst the erudite community and the building community. Though generally hostile, sarcastic or ironical, the latter cannot help but show a certain admiration. The subject is therefore an ideal observation point to reveal the quarrels and debates that opposed advocates of practice or theory throughout the 18th and the first half of the 19th century in Europe. The Tredgold aphorism according to which «The stability of a building is inversely proportional to the science of the builder»[24] is a good gauge of the manner in which the first essays on the mathematics of statics were perceived. The title of Charles-François Viel's memoir, entitled *De l'impuissance des mathématiques pour assurer la solidité des batimens*[25] *(Of the powerlessness of mathematics to ensure the solidity of a building)* is another.

While the theory on the mechanics of vaults progressed, stereotomy disappeared almost totally from architectural construction, mainly because of the

arrival of new materials such as iron and concrete. But the building of railways in Europe was to bring about the construction of relatively specific civil engineering structures, namely «oblique bridges» or bridges the deck of which is not perpendicular to the railway line. Such bridges have to withstand overloads and strong vibrations produced by the passage of trains whilst being —sometimes quite substantially— skewed. Their construction in stone therefore requires the resolution of delicate installation problems in order to reduce the outward thrust. Much has been written about oblique stone bridges. This literature is particularly interesting in that the civil engineering constructions concerned require that the problem be mastered both from the point of view of geometry and statics. In addition these writings are significant because they deal with an issue which needed to be addressed almost simultaneously all over Europe thus allowing a comparison of national characteristics.[26]

Stereotomy occupies a strategic position between geometry and mechanics. This explains on the one hand why it became a power stake and therefore a source of conflict between the different building guilds and, on the other hand the reason it played —and possibly still plays— a role in the formation of the various building trades.

Stereotomy as the scene of social conflict

Being the favourite scene of the practical versus theoretical debate, stereotomy has been from the Middle Ages right through to the 19th century the main stake in the rivalries between master mason architects and engineers.[27] This is so even though the terms of the practical/theoretical debate evolved considerably over such a long period of time.

It is to enable the architect to «direct and train master masons and their workers rather than be trained and led by them»,[28] that Philibert de l'Orme inserted the first treatise of stereotomy in a treatise on architecture. The Desargues-Curabelle quarrel, which opposed in the 1640's one of the most famous fitters with the best geometer of the time, is also a symptom of the tensions that existed. The essence of the Desargues-Curabelle quarrel bore on the manner in which one might ascertain that the lines of a drawing are legitimate. For Curabelle, the criterion was feasibility whereas Desargues only counted on whether the geometrical reasoning was correct. Now in this opposition, it is the entire status of the working drawing that is being questioned. If one admits along with Curabelle that the legitimacy of the drawing can only be validated by its execution, the master mason remains the keystone on the construction site. If, on the other hand, a drawing can, as Desargues claims, find legitimacy in itself, if its correctness can be shown purely on theoretical grounds and independently of any concrete execution, if optimal lines are found solely on the basis of geometric reasoning rather than experience, then the very status of the drawing becomes modified and hence that of its author and executant. Like de l'Orme, Desargues explained what is at stake in these conflicts: «just as Doctors of Medicine neither attend the schools or lessons of Apothecaries . . . neither should geometers attend the schools and lessons of Masons but, on the contrary, Masons should attend the schools and lessons of geometers, which is to say that Geometers are the masters and Masons the disciples».[29] Thus the salient feature in Desargues polemical writing is the assertion that theory takes precedence over practice.

With the emergence of the architects' guild and later on the guild of engineers, this current aims at increasing task specialisation. This clearly goes against the will of the stone cutters' guild that wanted to keep full mastery of the entire production process in complex vault construction. Thus, as the history of stereotomy unfolds, so does the history of the emergence of the different construction guilds and the way the territory was eventually to be shared between them.[30]

However, one can also have a differential reading of this history and try to understand how the actors are going to reappropriate their own history. We are now talking about the history of the history of techniques. For the history of techniques plays a social role and, in the conquest of professional hegemony, it is possible to find arguments in favour of the social division of labour and the means of legitimising its foundation. Rather than perceiving the tradition of guilds as resulting necessarily in routine behaviour that the use of geometric and statics theory might be susceptible of breaking (as argued by Desargues, Frézier and other engineers of the 19th century), the stone cutters' guild is seen as having a tradition which, far from being synonymous with

refusing change or observing stupid and immutable rules, represents the conscience of being a structured entity.[31]

Didactic Stereotomy, Theoretical Stereotomy

One last consequence —fundamental for its evolution in the 19th century— of the narrow and specific links binding stereotomy and geometry, is the transformation of stereotomy into a school discipline. It is of course not fortuitous that such a shift should occur at the time of creation of the Ecole du Génie de Mézières, one of the first engineering schools in Europe. Right from its creation in 1748 (thus before the arrival of Monge), the teaching of stereotomy in the school went beyond the strict utilitarian aspect of an already declining construction technique. The major objective of the course was to provide training in geometry and the art of visualisation in space. The founders of the Ecole du Génie de Mézières formulated this idea quite explicitly: «these arts offer such exact and precise knowledge for the drawing of plans and profiles, and the manner in which to express the relief they are to represent, that they may be regarded on the same level as the Elements (of Euclid)».[32]

This situation therefore opens a new area of research: stereotomy as a school discipline. What was its role in the training of engineers in Europe from the middle of the 18th to the end of the 19th century, how was it was taught and how did this teaching evolve . . . Since the history of learning institutions is dependent upon the history of the disciplines that are taught in them, the teaching of stereotomy becomes one of the possible markers for comparing institutions. Finally and for the same reasons, stereotomy also became part of the training imparted to workers and craftsmen, there again to a much larger extent than its strictly practical aspect would lead one to suppose. Indeed this is still so today if we consider the overemphasis of stereotomy in the training of the Compagnons de France (highly skilled craftsmen).

The fact that Monge, when an Ancien Régime engineering school was being transformed into an engineering school of the Republic, tried to transfer to descriptive geometry the didactic function previously assigned to stereotomy offers another reading of the relationship between the two disciplines.

Projective geometry, which decisively broke away from the graphical techniques out of which it was born, differs fundamentally from descriptive geometry, which remained linked to them body and soul. Poncelet did not propose a new form of pictorial art, give advice to painters or have artistic pretensions. Monge, in his courses at the Ecole Polytechnique, developed a stone vault construction theory based on curvature lines. This theory is extremely elegant from a geometrical point of view and is supposed to answer a relatively simple question: how can we generalise to any intrados surface starting from the case of the hemispherical vault? Yet one cannot but recognise that Monge's theory, which is based on curvature lines, is in fact for him a means of teaching the concepts of curvature lines and normal surfaces rather than teaching stereotomy. In any case, Monge finished off at the Ecole Polytechnique what the founders of the Ecole de Mézières had undertaken, by establishing what one might call «theoretical stereotomy», detached from its original function as a technique of construction and in radical opposition to the stereotomy of the work site.[33]

Conclusion

It appears to me the diversity of possible approaches is what gives stereotomy its specificity and importance as an object in the history of construction. Studies on stereotomy have successively been conducted by historians of architecture, historians of science and technology, and historians of education, and have benefited from the development of each of these particular points of view. But the true riches of stereotomy come from its intrinsic complexity and the constant interactions between historians from different fields. Therefore, the primary aim of studies on the history of construction today is more to decipher specific histories which interact at any given time, and put the pieces of a multi-faceted history together rather than explore a previously identified niche or other. The number and diversity of communications that follow under the heading *stereotomy* illustrates this wealth and polysemy.

I should like to conclude in a more personal way.

Like many other researchers present here today, I teach in a School of architecture. Now I do not perceive my activities as teacher and researcher as fundamentally separate. This is so not only because I teach history of construction but also because the history of construction in itself is for me a tool to vitalise teaching in schools of architecture. Stereotomy is a means —as I have just attempted to demonstrate— of approaching the history of architecture, understanding the mechanics of natural phenomena and learning geometry. Because of this and because its virtues as a school discipline do not appear to me to have entirely disappeared, I have given my students exercises in stereotomy for a number of years. The opening of the «Grands Ateliers de l'Isle d'Abeau» (near Lyon) in December 2001 allowed me to propose for the first time last year a stereotomy experiment on a large scale. I constructed with a group of some fifteen students and with the help of a professional stone cutter a planar vault of about 2.5 by 2.5 metres (see figure below).

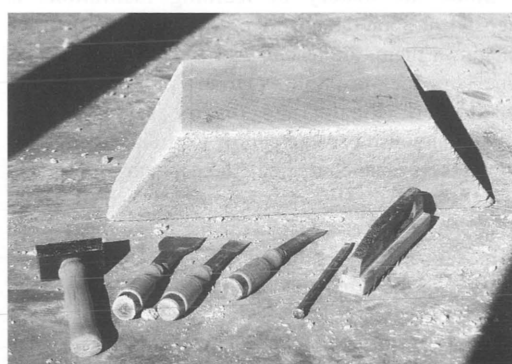
Leroi-Gourhan has written: «it would be of little importance that this organ of fortune that we call the hand disappear where it not for the fact that everything points to its activity being closely bound with the equilibrium of cerebral territories associated with it . . . to not have to think with all of one's fingers is equivalent to missing part of one's normally phylogenetically human thinking ability. There exists therefore as of now and at the individual level, if not at the level of the species, a hand regression problem».[34]

In carrying out stereotomy experiments on a large scale, the idea is to sensitise students to the complexity of the building endeavour. Given the relationship between stereotomy and the different branches of architecture I have just described, such experimentation is a means of learning «to think with all of one's fingers» about the history of architecture, construction, geometry and statics. In a time when the Internet imposes a rhythm of exchange on the order of immediacy, the practice of stereotomy, a hymn to slow motion, is I believe indispensable in teaching. It can renew the ways in which to apprehend fundamental disciplines in the teaching of architecture and to understand the relationships between them.

CONSTRUCTION OF A MORTARLESS KEYSTONE PLANAR VAULT IN THE GRANDS ATELIERS DE L'ISLE D'ABEAU (ISÈRE, FRANCE) FROM 4TH TO 8TH FEBRUARY 2002

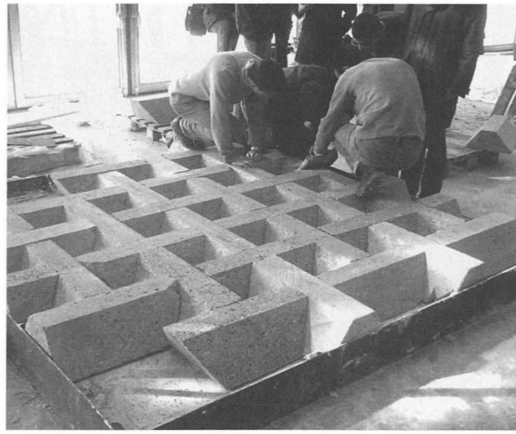
Under the direction of Joël Sakarovitch, Jean-Paul Laurent (structural engineer) Jean-Paul Foucher, (stone cutter).

Cutting of voussoirs

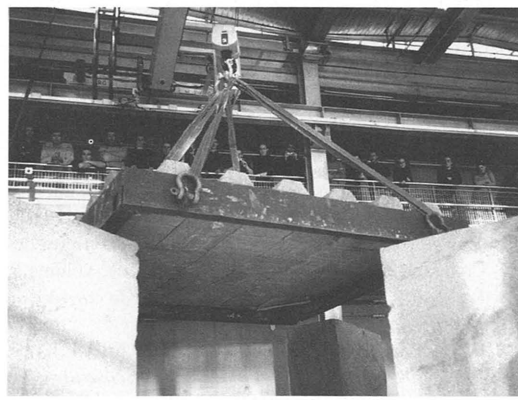


Voussoir and cutting tools

Assembly of first voussoirs



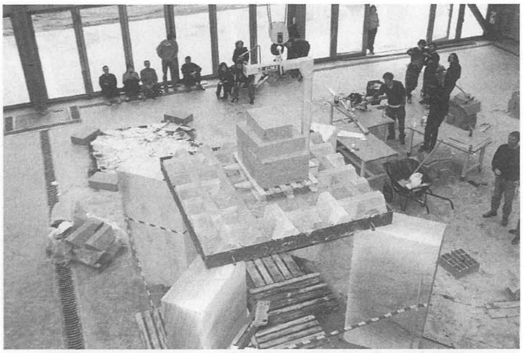
Voussoirs in their metallic frame

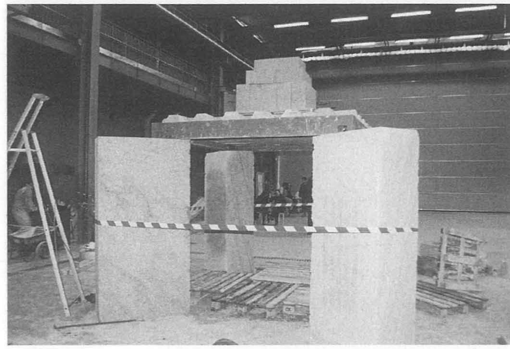


Positionning on piers

The planar vault

The underside





Load testing

NOTES

1. J.-M. Pérouse de Montclos, Ministère des affaires culturelles, Paris, 1972.
2. This definition is attributed to Claude Perrault by J.-M. Pérouse de Montclos in *L'architecture à la Française, XVIe, XVIIe, XVIIIe siècle*, Paris, Picard, 1982, p. 85.
3. On the subject, the following works are of interest: l'*Encyclopédie des métiers, La maçonnerie et la taille des pierres*, edited by l'Association Ouvrière des Compagnons du Devoir du Tour de France, volume 5, *Les outils* ; volume 6, *Le savoir faire (De la carrière au chantier)*, Paris, 1991–2000.
4. For a compete bibliography see A. Becchi et F. Foce, *Degli archi e delle volte. Arte del costruire tra meccanica e stereotomia*, Marsilio, Venezia, 2002.
5. See in particular Adam, J.-P., *La construction romaine, matériaux et techniques*, Paris, Picard, 1984 ; Ward-Perkins, J. B., *Roman architecture*, Electa, Milano, 1974; Kubach, H. E., *Architecture romane*, Electa, Milano, 1972.
6. One of the last is from Trevisan, C., «Sulla stereotomia, il CAD e le varie trompe d'Anet», in Migliari, R. ed., *Il designo e la pietra*, Gangemi, Roma, 2000, pp. 27–53.
7. On Spanish stereotomy, see Rabasa Díaz, E., *Forma y construcción en piedra. De la cantería medieval a la estereotomía del siglo XIX*, Akal, Madrid, 2000 or Palacios J.C., *La estereotomia en las construcciones abovedadas*, Instituto Juan de Herrera, Madrid, 1999.
8. Pérouse de Montclos J.-M., *L'architecture à la Française*, op. cit.
9. The serious foundation problems of the Maulnes pentagonal castle in Tonnerrois are probably an example of this rapture affecting Philibert de l'Orme
10. I'm borrowing this formula from Hubert Damisch, who used it concerning *L'origine de la perspective*, Paris, Flammarion, 1987, p. 86.
11. See Evans R., *The projective cast ; architecture and its three geometries*, MIT Press, Cambridge (Mass), 1995 or R. Migliari, op. cit.
12. See Shelby, L. R., «The Geometric Knowledge of the Medieval Master Masons», *Speculum* 47, 1972, pp. 395–421 or Recht, R., ed., *Les bâtisseurs des cathédrales gothiques*, catalogue of the Strasbourg exhibition, Strasbourg, 1989 and in particular the article by Müller, W., «Le dessin technique à l'époque gothique», pp. 237–254.
13. Dupin, Ch., *Eloge historique sur les services et les travaux scientifiques de Gaspard Monge*, Paris, 1819, pp. 20–21.
14. Olivier, Th., *Traité de géométrie descriptive, théorique et appliquée,* Paris, 1843, p. VII. However, in 1771, Tinseau presented a memoir to the Academy, published in 1780, where he clearly uses a descriptive geometry drawing.
15. See for example, the reedition of Philibert de l'Orme by Perouse de Montclos, of Vandelvira by Barbé-Coquelin, of Martinez de Aranda in the Biblioteca CEHOPU, the works of Müller, W., «Guarini e la Stereotomia», in *Guarini e l'Internazionalità del Barocco —Atti del Convegno*, Turin, 1970, vol. 1, pp. 531–556, the book by Bonet Correa A., *Figuras, modelos e imagenes en los tratadistas espagnoles*, Alianza, Madrid, 1993 or the article by Calvo Lopez J. and Rabasa Diaz E., «La coupe des pierres dans l'Espagne du XVIe siècle : le manuscrit de Gines Martinez de Aranda» in Becchi A., Corradi M., Foce F., Pedemonte O. (eds), *Towards a history of construction. Dedicated to Edoardo Benvenuto*, Birkhäuser, Basel, 2002 . . .
16. Lalbat, C., Margueritte, G., Martin, J., «De la

stéréotomie médiévale, La coupe des pierres chez Villard de Honnecourt», *Bulletin Monumental*, vol. 145, IV, 1987, pp. 387–406 and vol. 147, 1989, pp. 11–34. or Bechmann, R., *Villard de Honnecourt. La pensée technique au XIII[e] siécle et sa communication,* Paris, Picard, 1991.

17. In Taton, R., «L'Ecole royale du génie de Mézières» in *Enseignement et diffusion des sciences en France au XVIII[e] siècle*, Paris, Hermann, , pp. 559–613, p. 595.
18. Loria,G. *Storia della geometria descrittiva delle origini, sino ai giorni nostri*, Milan, 1921 ; Sakarovitch, J., *Epures d'architecture, de la coupe des pierres à la géométrie descriptive, XVI[e]-XIX[e] siècles*, Birkhäuser, Basel, 1998.
19. Michel Serres uses this expression concerning the origins of Euclidian geometry in *Les origines de la géométrie*, Flammarion, Paris, 1993, p. 209.
20. Quoted in Taton, R., *L'Œuvre mathématique de G. Desargues*, Vrin, Paris, 1988 (2[e] éd.), p. 240.
21. Benvenuto, «Résistance des matériaux (histoire de la)», in *L'art de l'ingénieur*, A. Picon, dir., Paris, Le Moniteur, 1997, p. 409.
22. However we have to notice that Leonardo is one of the first to propose a mechanical approach of the problem.
23. Cf. for example Benvenuto, E., *An introduction to the History of Structural Mechanics*, Springer-Verlag, New-York, 1991 or the works of J. Heyman and in particular «The stone skeleton», *International Journal of Solids and Structures*, vol. 2, 1966, pp. 249–279 and *Coulomb's memoir on statics. An essay in the history of civil engineering*, Cambridge University Press, Cambridge, 1972.
24. Thomas Tredgold , *Practical Essay on the Strength of Cast Iron and other Metals*, 1822.
25. Published in 1805
26. See in particular A. Becchi et F. Foce, *Op. cit.*
27. See *Dessin-Chantier : Stéréotomie*, n°1, Grenoble, 1982.
28. L'Orme, Ph. de, *Nouvelles inventions pour bien batir à petits frais*, Paris, 1561, reed., L. Laget, Paris, 1988, p.38.
29. Desargues, «Reconnaissance de Monsieur Desargues» in Bosse, A., *Manière universelle de M. Desargues pour pratiquer la perspective par petit-pied, comme le géométral,* Paris, 1648.
30. On this subject see Potié, P., *Philibert de l'Orme, figures de la pensée constructive*, Marseille, Parenthèses, 1996.
31. See Assegond C., *Socialisation du savoir, socialisation du regard. Les usages sociaux du savoir géométriques et de la stéréotomie chez les compagnons tailleurs de pierre*, Université de Tours Thesis (France), nov. 2002.
32. Quoted in Belhoste, B., «Du dessin d'ingénieur à la géométrie descriptive», *In extenso*, n°13, juil. 1990, p. 111.
33. This work site stereotomy is very well described in Rabasa Díaz, E., *op.cit.*
34. Leroi-Gourhan, A., *Le geste et la parole*, t. II, Albin Michel, Paris, 1965 p. 61–62.

Modern analysis and historic structures

David Yeomans

Some time ago I discovered a teaching film that featured a guide to one of the French cathedrals demonstrating the action of the vaults and flying-buttresses to parties of tourists. This he did by lining up the men in two rows to represent the columns of the nave. They then raised their arms to represent the ribs of the vaults, linking hands with their neighbours and those opposite at the crown. Their wives then stood behind with their hands on their husbands' shoulders to be the buttresses. The guide then placed his hands over the «bosses» at the meeting of the ribs and lifted his weight off the ground hanging from the outstretched arms of the men. The result was that all could feel the forces thus created. Of course we may object that this is not the loading pattern on the vaults and so is a rather inaccurate model but the demonstration served the purpose for the visitors who surely would never forget this little lesson in structures. Also, when I introduce structures to students I do so by pointing out that the image of structural analysis is that it uses a large number of formulae but for me the most important formula for them to understand is:

Structural calculation = Guess.
It is just that some guesses are better than others.

All structural calculations depend upon building some model of the situation that one is trying to understand and the model that one uses has to be appropriate for the task in hand providing a sufficient degree of accuracy in its representation of both the structure and the loads that come onto it. What being «appropriate for the task in hand» means is that it helps to answer the questions that are being asked. Thus being clear about the questions one wants to ask is the first step in the process. But it often seems that this step has been missed out in a rush to calculation? My motto is «Work is no substitute for thought» and this is particularly true when modelling an existing structure as distinct from one that only exists on paper because the two processes are not the same and neither are the purposes for which the analysis is carried out. Many carrying out the analysis of existing buildings seem not to recognise this simple fact and it is also sad to note that there are some who approach the task without looking at all closely at the structures they are purporting to describe.

I would like to address these issues asking how the purpose of calculation affects the methods used and particularly how this affects the historian. In doing so I shall need to consider how some earlier work satisfies my criteria and this will mean directing some brickbats at some distinguished scholars. But I take the view that life would be very boring if we all agreed with each other.

The reasons for analysis

The obvious starting point is to consider the different kinds of people who carry out the analysis

of historic structures and we can identify three kinds: conservators, historians and those academics who do not properly fit in either category. However it is also useful to consider the designers of new structures for the purposes of comparison because most analytical techniques are devised with this task in mind. The designer of a new structure is attempting to produce a design that will be safe and to do so with reasonable economy both in the structure itself and in the design process. This is also the task of the conservation engineer although with the additional requirement to respect the historic fabric.

Of course much of the early work done on the analysis of historic structures was done in the course of restoration work. Well known historic examples of this are the eighteenth-century analysis of the dome of St Peter,'s, Rome and Baines's analysis of the roof of Westminster Hall, London in the last century (Baines 1914). The purpose of such studies was and still is to guide possible strengthening or repair work and the methods need to be appropriate for that. But as historians we have different questions to ask and it is important to be clear about what they are. It is also important to be clear that any calculations carried out both by conservators and historians differ from those used in the design of a structure. Thus there are three different situations and so three different kinds of questions that might be asked and three different approaches needed towards analysis. Although I am not concerned here with calculations carried out for repair purposes it is difficult to ignore these completely in my comparisons.

While both deigners of new structures and conservators are concerned with the safety of a structure this is not the concern of the historian. For the historian any analysis is surely carried out to answer one of two kinds of question:

1. How was any particular structure or type of structure designed?
2. How did the behaviour of a given type of structure affect the design of buildings for which they were used.

These will lead to subsidiary questions that we will come to shortly but these seem to me to be the fundamental issues. But we also need to consider the academic who may have neither the safety concerns of design nor the questions of the historian. Nevertheless the results of such work might well be of interest to one or the other.

THE NATURE OF THE PROBLEM

It is in the nature of the problems presented to these different people that determines the suitability of the methods available to them; the nature of the problem being a combination of the questions that are asked and the practical problems presented by the analytical task. The intention of the designer has been clearly stated by Heyman in his discussion of Westminster Hall roof. It is to demonstrate that there is a «distribution of forces (in the structure), in equilibrium with the external loads and not exceeding the. . . limits for individual members». If this condition is satisfied then the structure will be safe and it does not matter whether or not it behaves exactly as predicted by the model (Heyman 1967, 159). Of course, the designer has the luxury of being able to specify the properties of members to ensure that this condition will be met.

This is not true of someone studying an existing structure. One cannot specify the properties of the members and one may not be able to measure them with any degree of accuracy. Therefore one has to devise methods that take into account the uncertainties that one has both about the nature of the assembly and the properties of individual members. Jacques Heyman's approach to historic structures, as stated above, has advantages in that he does not need to know the properties of all of the members in the actual structure, simply those of the modelled structure. Thus for Westminster Hall roof could ignore the properties of the arch brace which was assumed not to be acting. But this overall philosophy hardly seems adequate for a conservator.

Suppose, for example, that I have a model of the structure that ensures its overall safety but that there is a member that is in fact carrying load although it is not relied upon in this model. Failure of the member will not cause collapse but simply a redistribution of the load in a way that accords more closely with my idealized model. This result would be a loss of integrity of the failed member and possibly some other damage or unacceptable deflections as loads are redistributed. It might also mean that the structure is no longer working as its designer intended. For these

reasons, as a conservator as well as an historian, I have some sympathy with Roland Mainstone when he asked of Westminster Hall roof how the structure was actually working (Heyman 1967, 788–92). A conservation engineer might need to know how the load is actually being transmitted in the structure because of a concern for states of distress within it and perhaps because of a need to replace some members. The problems being faced may also involve determining loads on temporary supports to be used in the course of restoration work.

For the historian.

While an historian who wishes to carry out an analysis of a structure has the same practical problems as the conservator there can also be the additional complication that the structure in question no longer exists. Having been demolished some time ago it might only be known through historic records that themselves may be of uncertain accuracy. This was Heyman's position when he carried out an analysis of the Cismone bridge (Heyman 2000) and it is true of a number of structures that I have been interested in; the roof structures of Inigo Jones for example (Yeomans 1986) while others, including Dorn and Mark (Dorn and Mark 1981) have considered the structure of Wren's roof for the Sheldonian Theatre, Oxford that was replaced in the early nineteenth century and for which the various drawings disagree. The reason for one's interest in such structures is that they might be significant to the development of structural deign or might lead to insights into the structural understanding of the time.

Both the two basic historical questions set out above seem to a number of subsidiary questions but I shall confine myself here to the first, i.e. How was any particular structure or type of structure designed? This applies either to the design of a single structure or to the development of a structural type and requires that we ask such questions as:

i) What was the general level of structural understanding at the time?
ii) How was this applied to a particular structure?
iii) Would feedback from the behaviour of the structure have modified such understanding?
iv) What understanding did the designers have of the materials they were handling?
v) Who actually carried out the design?

I don't pretend that this is a comprehensive list but it shows the range of questions that might be asked.

Some of these can be answered from documentary sources but our concern here is those for which structural analysis is of some help. If we are trying to put ourselves in the mind of the designers of the time then until the eighteenth century these would have had only very simple ideas of structural behaviour. Moreover much structural design would have been carried out by men of limited scientific or mathematical knowledge. Methods of design depended upon different kinds of rules but might have been modified by observation of the behaviour of structures. My instinct therefore is to assume that one should if possible look at the documentary sources for the way in which structures were deigned and to structural analysis for their behaviour and so for ways in which designers might have learnt from that. This means that timber and masonry structures present quite different problems because of the very different nature of the two materials.

Unless there are problems of foundations settlement, which of course was not unknown, masonry structures tend to fail catastrophically so that there is little sign of distress until the building is close to failure. There is certainly no indication of the level of stress within the structure that could act as a guide to improvements in design. Thus, I must take issue with Robert Mark's apparent assumption that because analysis of a sequence of the structures of French cathedral shows improving structural efficiency the designers were somehow aware of this and were consciously developing their designs for that reason (Mark 1982). However, his demonstration that Chartres would suffer distress at a high level without the upper flying buttresses and his subsequent observation of cracking in this area shows that while his model technique is not strictly correct, it can provide useful insights. It had the effect of directing attention to that part of the structure that would have exhibited movement that could be observed by the builders of this time and responded to (Boug & Mark 1973).

Naturally I agree with Huerta (2001) that the «equilibrium approach» is best for the analysis and design of masonry structures and also that Mark's

application of photoelastic methods is strictly incorrect, in that we are neither dealing with an elastic material nor with one that can take tension. Nevertheless, the above example shows that his photoelastic approach does have some advantages the study of Chartres being an example where the method was sensibly applied to a particular problem. But this particular analytical technique can say nothing about the general level of structural understanding at the time these buildings were constructed nor about how the designs were developed.

The other limitation of this method, and others that simply consider the final structure, is that they fail to consider the whole process that the designers of masonry structures would have been only too aware of. The problem was not simply to ensure that the completed structure would stand but to be able to get it there in the first place so that questions of setting out and of temporary supports during construction would also have taxed the minds of the designers. If we are trying to understand how designers behaved in the past then surely we must be trying to create in our minds the conditions that they observed when they built and loaded their structures so that we can «observe» what they observed. How did this structure behave when built? How were the components of the structure placed in position? What was the sequence used? All of these are questions that involve understanding the behaviour of the structure during construction. These were the questions behind Mainstone's study of St Maria del Fiore (Mainstone 1969/70) but unfortunately we have precious little evidence for construction processes. Even the most celebrated buildings have left little in the way of documentary evidence for this so that we are left with a certain amount of speculation. Mainstone's study showed how the Brunelleschi dome could have worked while a Heyman demonstrated the form that flying buttresses must take in order to cope with two extreme states: their own self weight acting alone and the thrust of the vaults. What neither studies tell us is how the designers of the time might have come to these necessary forms.

TIMBER STRUCTURES

It is in timber structures that we are likely to see the kind of behaviour from which later designers might learn because of the extent of movement in the material. Timber structures also offer the advantage that collapse would seldom be catastrophic but be preceded by a period when distress could be observed and during which remedial action might be taken. Of course movement of the timbers is as much associated with drying shrinkage and creep deflections as it is with elastic deformation. Moreover the behaviour of various kinds of framework might also be affected by the accuracy of the carpentry. It is these uncertainties in the behaviour of timber structures that makes one question the validity and usefulness of many of the sophisticated methods of analysis that have been used and it only takes a simple example to demonstrate this.

The effect of shrinkage of the timbers is most clearly seen in the so-called clasped purlin roof, typical of English medieval construction. The sequence of assembly of this structure suggests the load carrying mechanism. Posts were stood on the tie beam to support a collar that in turn supported the purlins. Finally the principal rafters were added clasping the purlin between them and the collar (Fig 1). Clearly the initial structural action on completion of the roof was that the purlin loads were be transmitted via the posts to the tie beam. (Fig 2a) However subsequent shrinkage of the tie beam commonly results in a gap between the bottom of the posts and the top of the beam. Obviously load is no longer being transmitted by this route and one must conclude that the purlins are now supported by arch action of the principal rafters and collar. (Fig. 2b) Analysis of such a structure must begin with observation of its present condition.

Considering the effects of shrinkage, creep and uncertain standards of craftsmanship together with an

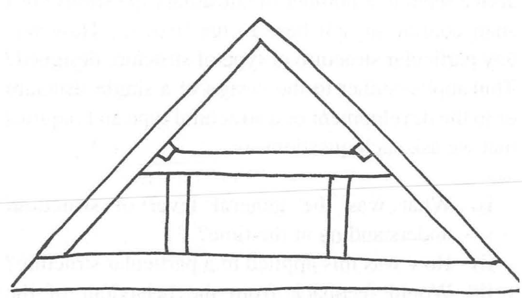
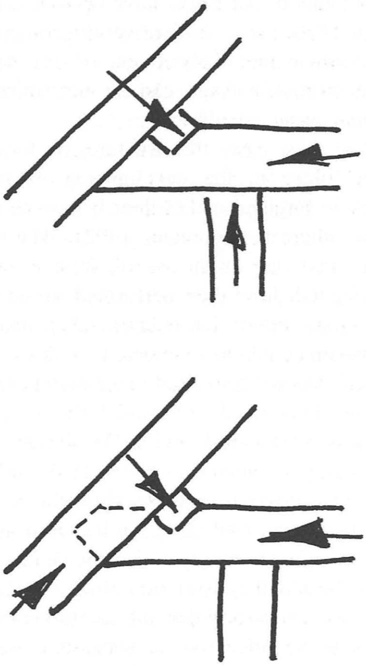


Figure 1



Figures 2a and 2b

imperfect knowledge of the properties of the timber sections makes any detailed analysis of the structure of very doubtful validity. I must echo Huertas comments about masonry structures when he opines that the very nature of the ashlar and rubble wall means that the use of sophisticated elastic models is contrary to common sense. Faced with this uncertain behaviour my approach to timber structures as a conservator is sometimes to consider quite different modes of behaviour for different members in the frame asking what range of forces I might have to deal with in making a repair. One can ask the same questions when considering the historical behaviour of the structure. A purlin might have carried loads from the rafters back to the supporting frame and this would determine the loads that frame had to carry when the structure was built. With time purlins frequently deflect and shed load back to the rafters that in turn delivered a larger load to the wall plate. How might this have affected the carpenters' approach to the design of such structures?

As with masonry structures, stresses in timber structures were generally low; what would have concerned their builders were deflections, particularly shrinkage deflections, as is clear from early carpenters' manuals (for example Nicholson 1792). Of course stresses would be high at joints and a concern for this is sometimes seen in their detailed design. For the 60ft spanning trusses over the portico of St Martin's in the Fields, London, James Gibbs had wedges driven into the head of the king post to tighten it against the ends of the principal rafters. This might not have done much to counter shrinkage effects but would certainly have taken up any initial lack of fit in the carpentry. The ends of posts or braces on long-span trusses might be notched into tie beams or principal rafters to take the thrusts from the inclined struts (Fig. 3). Without such details these thrusts would have been taken on a comparatively narrow tenon so that it is clear that their designers recognized the nature of the forces involved. But could the stresses on the tenons have been sufficient to produce noticeable stress, i.e. to have resulted in some crushing of the timber in long-span trusses? Here is a question that asks for some analysis in order to

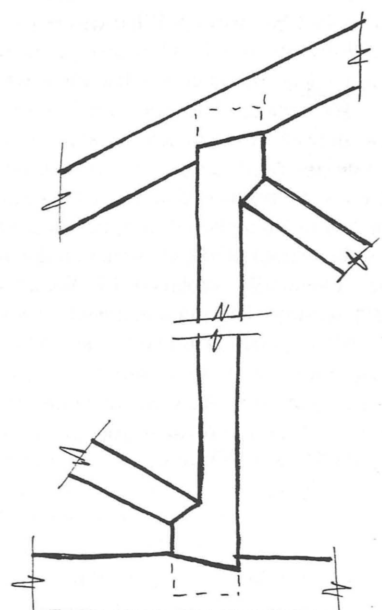


Figure 3

answer it —but how detailed need this analysis be? Assuming that the tie beam is in two lengths, which was common at the time, a simple king post truss is indeterminate because of the continuity of the principal rafter. But if the principal rafter is strutted so that there is no deflection in it then we can reduce the structure first to the principal rafter as a continuous beam over rigid supports to find the support force and then the roof truss to a pin jointed frame. This will then give the maximum forces that could be generated in the strutting members and would be sufficient to give an indication of the joint stresses. By this simple means one can have an answer to the question. And note how the question was phrased: it was *could* the stresses have been high enough to produce crushing of the timber not *would* they have been high enough. Thus an upper bound for the value of the stress is appropriate.

FAILURES AS A MEANS OF FEEDBACK

One obvious method of learning from timber structures is the observation of distress in structures and some of the early roof trusses built in England did indeed give trouble, presumably because of either poor design or poor execution. Inigo Jones's roof of the Banqueting House in London gave problems from a very early date, so much so that for a while the interior of the building was scaffolded to give some support to the roof. Drawings made at the time of its construction, or shortly after, show a poor design of timber truss with iron bars that may have been added in an attempt to strengthen it, but these are not clear enough to be certain of the construction details. The roof was eventually replaced by Soane in the nineteenth century. The same architect's roof for St Paul's, Covent Garden also had timbers added in an attempt to improve it as shown in contemporary drawings but this structure was eventually destroyed by fire. One of the puzzles about the roof of the Sheldonian Theatre, Oxford, an early deign by Christopher Wren, is the reason for its early nineteenth century replacement – but here we do have some reasonable drawings. This was an ingenious piece of carpentry that was much admired in its day for the design of the tie beam that comprised seven timbers joined together to transmit the tension force (Plot 1677). However there was some early concern for its deflection that might have been as a result of shrinkage of the posts. As the overall geometry of the roof is known, the likely extent of any deflection resulting from shrinkage can be determined from some fairly basic calculations.

In all of these cases the structures no longer exist but their place in the development of structural carpentry in England makes their behaviour a matter of some interest (Yeomans 1992). The kind of questions that one might ask of these is how they were designed, how they performed in service and what lessons might have been taken from their behaviour that could have affected the design of later structures. Any analysis used in an attempt to answer these questions needs to model the structure as realistically as possible and to be clearly directed towards the phenomena in question. Consider these requirements in relation to the photoelastic analysis of the Sheldonian roof structure by Dorn and Mark (1981) and it is seen to say nothing useful, nor does it consider the actual construction of the structure.

I am also concerned that the questions raised are pertinent to historical issues because it sometimes seems that they are asked simply as an excuse for the analysis. Morris and his co-authors (1995) recently contributed a paper to those that have already discussed the roof structure of Westminster Hall, London. They set out to discover the extent to which the decorative tracery might be contributing to the behaviour of the structure. This was an interesting exercise using finite element analysis but leads me to ask the «so what?» question. It is difficult to imagine that the builders of this roof, or even of a host of smaller related roofs, could have imagined the tracery to be part of the structure. Nor if it were part of the structure in this case that it would also be so a sufficient number of other cases to deceive the carpenters about the structural action of the roof and so affect the development of the type. Therefore the result, whichever way it went, could add nothing to our understanding of the history of this particular roof's design or that of any other even though it might say something about their long-term behaviour. But most of all I doubt the usefulness of finite element analysis for the understanding of any timber structure on the common-sense grounds that I have already adumbrated.

It should be clear by now that my concern is for clear historical questions if possible addressed

through the simplest methods of analysis possible. There are several advantage of using simple techniques. The first is that they are quick and easy to carry out. The second is that one has a clearer idea of the meaning of the results and particularly of the effect of any simplifications that are made. The third is that simple methods might possibly be understood by a reasonably numerate architectural historian which means that the historical ideas are more easily communicated.

Of course this third reason could seen as a disadvantage. Perhaps it gives people a good feeling to be privy to a mystery that others do not understand, to be able to play the priest among the laity. I am reminded of Bernard Shaw who said «All professions are conspiracies against the laity». Thus when the conservator engineer uses finite element analysis with its wonderfully colourful print out of the results is must surely impress the client. If I don't understand it, and especially if I have no hope of doing so, then it must be good. And if I use the same techniques in an historical context and the historians cannot understand it then it makes me master of the field. This is not something that I subscribe to because in both areas I would be concerned to facilitate communication and mutual understanding. This means that one of the factors for selecting a particular method of analysis is because of its ability to facilitate communication with others. This brings us back to the tour guide's demonstration; structurally inaccurate but effective for its purpose. In a similar way Mark's photoelastic models are popular with students because of their very graphic nature. Better still are Huerta's reduction of Heyman's approach to a demonstration using cardboard models because they are not only as graphic as Mark's models but are a more accurate representation of reality and have the added advantage that it is something that students could do for themselves.

THE ACADEMIC ANALYSIS

There are those whose purpose in analysing historic structures seems to go little further than the analysis itself and thus some explanation of observed phenomena within the structure. Of course this is a perfectly respectable academic exercise and there are those who have built distinguished careers on just this. Moreover such studies can also provide valuable information for the conservator. For example, Heyman's analysis of masonry vaults shows patters of cracking within them are perfectly normal and need not be of concern. But in doing so they avoid asking any real historical questions. It would be fair to say that while Heyman is interested in the history of structural analysis, as seen his recent publications (Heyman 1998), he would not consider himself an architectural historian and so does not always ask the historical questions suggested by the structures that he analyses. An example here is his recent paper on the behaviour of the Cismone bridge as illustrated by Palladio (Heyman 2000) He does not, for example, consider that the tension forces might have been carried by a metal strap, as suggested by the drawing, nor does he consider the relevance of Inigo Jones's sketches of connection details on this page of his own copy of Palladio's book (Allsop 1970)

I am reminded of the study by Eda Kranakis who showed how the different approaches of Finley and Navier to the design of suspension bridges were affected by their social circumstances (Kranakis 1997). She shows that Finley who was concerned to patent a system of design that could be used by others for the construction of relatively cheap bridges developed a system that relied upon a simple model analysis. In contrast, Navier, who worked within the career structure of the Corps de Ponts et Chauses was anxious to demonstrate his ability to carry out a thorough mathematical analyses that dealt with a range of questions irrespective of their practical value in design. Is it not equally appropriate to ask about the social circumstances of those who carry out analyses of historic structures? It seems to me that there are a number of examples that use sophisticated techniques, including finite element analysis, in circumstances where they appear to be of dubious historic value. Presumably it is the position of their authors within engineering departments that inclines them not only to ask purely structural questions but also to use the most sophisticated techniques available to them. This is where I divide from many engineers who turn their attention to historic structures because I want to place their analysis in a wider context. I want to consider how social and economic considerations affect the design and wish to use analysis as an aid to answering these questions and not simply an end in itself. Nevertheless some of

the remarks that I make about the appropriateness of various techniques are equally applicable to the more limited exercises.

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Communications

A peculiar architecture: The open staircase of Naples

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Lucia Bove
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Maria Lippiello

This is a summary of a much wider research that, starting from a specific aspect of the architecture of Naples of the 17th and 18th centuries, aims to find common roots between Neapolitan technicians and scientists and other scientists in Italy and Europe; it seeks to understand if and to what extent new sciences influenced consolidated techniques and how the social, economic and political context speeded up or slowed down new developments. In this paper, we study in particular the Neapolitan open staircase in minor architecture, which borrowed from famous examples what have become common features of the buildings of the city.

The Neapolitan open staircase

The enduring Greco-Roman urban layout in the old town center of Naples and therefore of the building lot generally closed on three sides has brought about the development of a particular type of construction that inherited the airing and lighting of the internal courtyard from the Roman *domus.*

In the second half of the 15th century, there was the Aragonese city wall and laws prohibited all *extra moenia* development. Population growth caused buildings to grow upward and the use of excellent volcanic tufa quarried *in situ* enabled buildings of the same lot, with constructions of maximum 2 storeys, to be raised to 4 or even 5 floors. The resulting high buildings are out of proportion with the streets, which totally coincide with the preexistent cardines and decumens, virtually preventing the development of a continuous street front.

Though Renaissance influence at times led architects to use some sort of rules, street fronts are generally fragmentary and hardly ever viewed as a whole: only the vaulted entrance and the portal display the pomp of aristocratic buildings.

This gives rise to a new element of architecture, another façade: the internal courtyard front facing the entrance at such a distance from the street axis as to allow the view of good part of it through the entrance hall; a new element to fit in the overall architecture. The internal yard, at times completed by loggias of medieval or Renaissance origin, acquires new elements: vertical connections.

This distribution typology, which becomes a feature of the city, isn't shown in a modern zenith map since the separation of the public spaces (streets) from the private (dwellings) can't fully represent the connections nor the real aspect; in ancient maps, for instance the one by the Duke of Noja (1750–1775), cross-sectioned at an height of 6 (*palmi*) spans (about 1,60 meters), where the city is drawn according to path contours, the interconnections between public and private spaces are reported and the latter are integrated in the general layout of the city thanks to the continuity of a wider visual space of the entrance hall opening out onto the courts (Fig. 1).

The typical Neapolitan staircase opening out onto the courtyard thus completes the façade with a series

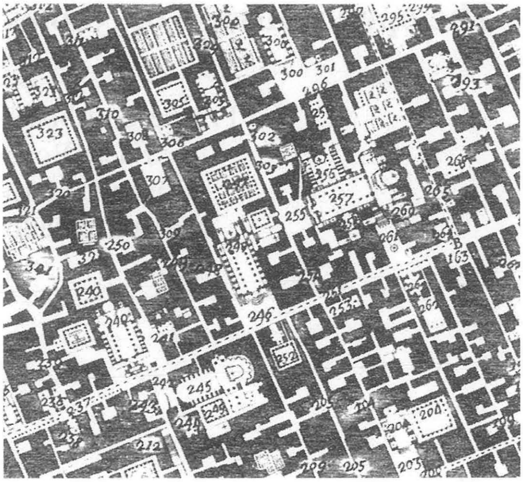

Figure 1
Duke of Noja's map. Detail

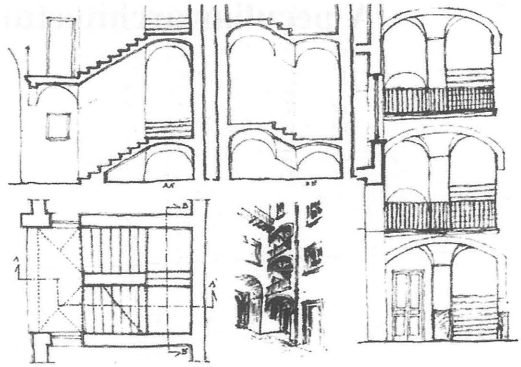

Figure 2
The staircase on central bearing wall

of screens from street to dwelling, from public to private spaces, a continuous filter created by the entrance hall-court-staircase system.

The first examples of Neapolitan open staircase from the Renaissance derive from the loggia plan and sometimes they are in fact set in loggias; the resulting flights of stairs, positioned at right angle to the yard, are borne by two staggered rampant barrel vaults jointed on the same bearing wall and cross-vaulted (or cap vaulted) at the level of the intermediate landing, paired up by a separating arch corresponding to the thickness of the wall. Level with the aspect on the court and with the landing leading to the two flats, we again find the same pair of vaults, whose arch is normally held by a monolithic pillar in piperno also used for facing the arches: a loggia with the same cadence at times on all levels is thus formed.

The staircase on central bearing wall

In this type of staircase developing on two flights on barrel vault, achieved through successive round arches built on a slanted plane resting unbroken on the external wall of the staircase and on the central bearing wall linked by paired cross or cap vaults with the necessary bearing points, a further transformation takes place: the elimination of the façade pillar generates a single segmental arch but with rise equal to the others and the consequent cap union between two half cross vaults or the deformation of two cap vaults jointed by a haunch. The central bearing wall staircase is usually found on the side front of the court, creating asymmetry between the two flats, only one with internal outlook and the other facing the street and with outlook onto the yard.

Though more convenient for the type of vaults used, this solution has the drawback of the length of the flights which, as in this case, are two. To solve this problem, some steps were often added at the central bearing wall level and it's this development that, maintaining the barrel vault, has led to the next type of staircase with three flights.

The closed stairwell

Unlike the staircase on central bearing wall which is always rectangular, this type has a planimetry divided in nine uneven squares with the central one, the stairwell, closed on four sides (the interior is generally inaccessible) to allow unbroken support for the three barrel flights whilst for the two vaults at intermediate landing level a bearing point is necessary, as in the previous type of staircase, with cross or cap vaults. The vault of the main landing overlooking the court is quite interesting since it rests on three squares: a single segmental arch overlooking the yard, two side distorted cross or cap vault sections

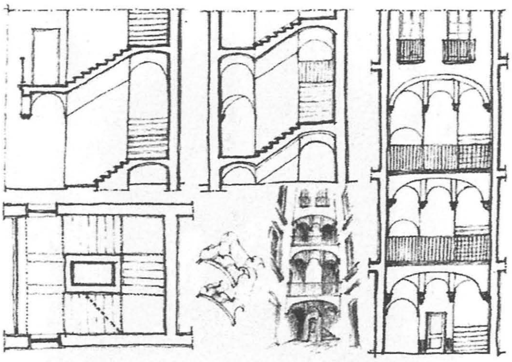

Figure 3
The closed stairwell

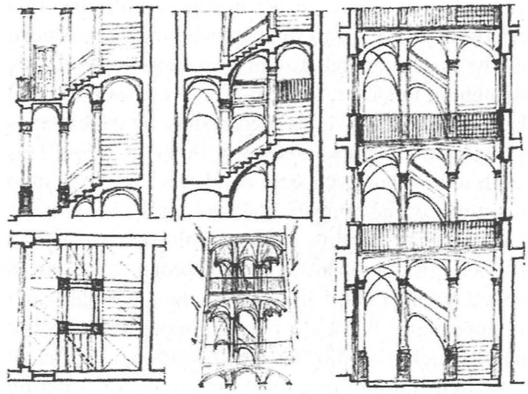

Figure 4
The stairwell on pillars

and a half barrel vault at the level of the well wall. Though cheap, this solution has an unsolvable drawback: the inner flight of stairs is badly lit.

The stairwell on pillars

The next step was knocking down the well and replacing the continuous wall with four pillars leaving the same layout of nine squares and three flights of stairs. The vaults had to rest on bearing points and the look of the flight support changes while the intermediate landings remain the same: rampant cross vaults (created by the intersection of a flying barrel and a bevelled barrel vaults) or rampant cap vaults (distorted cap vaults to create bearing points at various heights). Obviously, the facing arches of these vaults are bevelled (goose neck) to joint the pillars; unlike the previous models based on continuous support, an almost medieval structure system of arches bearing the vaults is created. Like the closed stairwell, the main landing too in this case develops on three squares, the side squares with distorted cross or cap vaults, whilst in the central one corresponding to the well, instead of the half barrel, there is a fanlight forming almost half a real cross vault and the other half is mock.

The segmental arch overlooking the court is thus charged athwart but its mass is still considerable enough, compared to the relatively small vaults, to ensure neutralization of the thrusts.

The open stairwell

The mass increase of the arch facing the courtyard making it look like a barrel vault, often with transom to create bearing points and spreading for the whole width of the landing, has permitted to bear the weight and the thrust of the flights achieved by slanted flat arches thus freeing the stairwell of all bearing structures: the open stairwell, flights of stairs on slanted platbands borne by the landing vaults loaded

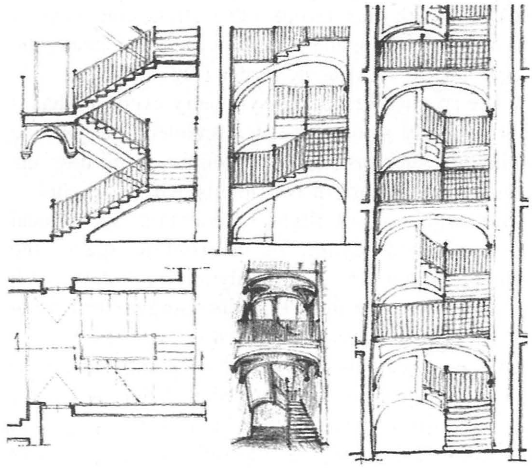

Figure 5
The open stairwell

athwart. This staircase often consists of only two almost adjacent flights of stairs with the well reduced to the minimum but it can also have three flights though, in this case, the rear arch becomes a beveled barrel vault with two landings resting on the extrados at different levels jointed by a flight of steps. This vault too is charged abeam and bears the weight and the thrust of the rampant platbands at the level of the landings. The need to joint the oblique platbands to the bearing arches has led to an improper vault, the so called rampant half barrel vault, which is a half vault in appearance but it's a flat arch along the directrix and as such it's built, though it follows a quarter of the arch in the opposite direction for jointing purposes.

Later developments

These four types of stairwell in masonry with infinite practical applications remain a typical architectural feature of Naples even when the city walls had become of no use and it was legal to build *extra moenia.* The 17^{th}–18^{th} century lot enclosed by streets includes an open space but the built-up area around it has developed according to a consolidated type of housing with the only difference that it has a previously absent rear façade for air and light and the staircase at the back of the courtyard has a view of the garden only if it is either borne by pillars or is open. If instead it's a staircase on bearing walls or, less often, it's a closed stairwell with side layout, other stratagems such as archways and loggias on the rear side create the possibility of viewing the garden (Fig. 6).

The strained search for symmetry even in what is not naturally symmetrical led architects to contrive solutions such as: space permitting, splitting the stairwell on pillars in two interpenetrated flights of stairs with a third flight in common, the typical pincer-shaped layout; in insufficient space, the structure often seems split only at ground level with an added opposite flight of stairs going no further but which is symmetrical to the first flight (Fig. 7).

Other stratagems were contrived to make up for the height difference of the intermediate landings. Unable to vary the corresponding planimetry of the wall structure and, even less, the ratio tread/riser, some steps have been placed, where necessary, at the level of the bearing wall thickness or curtail steps added

Figure 6
The loggia

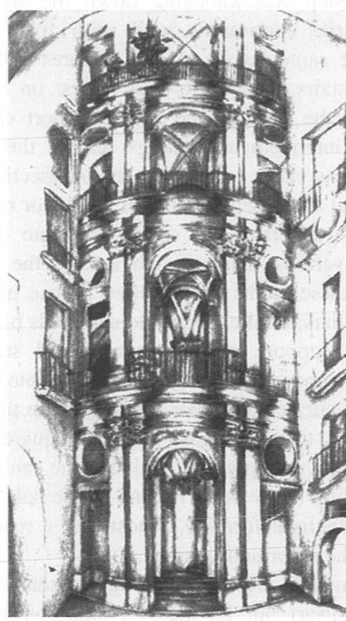

Figure 7
The stairwell on pillars

outside the closed well or well on pillars where interpolation is not at all possible.

Since vaults need provisional structures in any case, these staircases are assembled so that part, if not all, of the centering was re-employed.

In staircases on bearing wall, the centering of the rampant barrel vaults is normally the same and was used twice for each level and the same for the centering of the intermediate landing vaults, whether cap or cross vaults. In the Renaissance solution with the two front arches and two equal landings, it could in fact be used four times. In the single segmental arch, however, it was possible to use half of the mould of the cap or cross vaults, given the characteristics of the main landing vault, preparing the division along the diagonal and re-assembling the centering on the other floors. It only involved adding to the prepared half centering the suitable barrel and cap joints. The same logic can be seen in all other types: the centering was reusable at the same level and in any case the added parts could be used again for each floor.

At this point, it behoves us to say a few words about the Neapolitan metric system and the laws of the time concerning cutting the tufa in specific sizes. The *Palmo Napoletano di costumanza* (The Neapolitan palm in use) corresponds to 0,26333 m (from 1480 to 1840) and the *Palmo Napoletano decimale* corresponds to 0,26455 m (from April 6 1840). There is reference to the *Palmo Napoletano di costumanza* in the *Pragmatica* of August 27, 1564[1] of the Viceroy Don Perafan de Ribera Duke of Alcalà where, amongst other important norms to eliminate frauds in the building industry, it's made compulsory for the *tagliamonti* (tufa quarrymen) to furnish tufa blocks in three standard sizes: the *pezzo* (1+½×1+⅓×½ *palmi* or spans), the *spaccata* (2×1+⅓×½ spans) and the *spaccatella* (1×1+⅓×½ spans) with two constant dimensions (a thickness of ? a span, equal to 13,16 cm and a height of spans, equal to 35,11 cm) and only the length varying (39,50 52,66 and 26,33 cm respectively) to ensure total modularity. Since these stones were fitted with minimum adjustment, the stressed surface remains constant according to the chosen size and, considering the load on these arches, it was more than enough for the purpose. Easy to saw, a property of the yellow variety of Campania, the tufa could be quickly transformed in tapered ashlars. The cutting operation created a lot of scrap material often employed as filling but also mixed with lime and pozzolana to make real concrete similar to the Roman *opus cementiciun* to build directly the landing and flight vaults. Therefore, in the stairwell structure, the arches and the vertical elements are the load-bearers of the main structure and the vaults, supported by the former, load-bearers of the secondary structure, given the relatively small dimensions. The vaults, however, often have a rough centering over which was poured the concretion to level the extrados. A rather light concrete was used made of lime, lapilli and sometime even lava foam (normally used especially as prop) and the already mentioned *mazzacani* (quarry rubble). In the open stairwell, however, the vault structure centering of the landings, due to the absence of main structures, is quite precise and similar to the slanted flat arch centering that bears the flights of stairs, the so called half-barrel flights.

Pragmatica also contains norms concerning the piperno quarriers, obliging them to furnish blocks according to precise standards for structural purposes and banning the use of this stone for sole decorative purposes. This stone is used for archways (moulded in position), often for pillars, for reinforcing openings such as doors and windows as well as for steps, because of a certain abrasion resistance. The steps are often made of single blocks which load on the arches, on the perimetrical wall and, where present, on the bearing wall, thus unburdening greatly the flight vaults. Slabs separating the tread from the riser have instead been used in open stairwells to lighten the oblique platbands.

Sciences in Naples

It's impossible to understand innovations and weaknesses in the scientific field of the city without a glance at the political situation and institutions of the time; if it's true that the 17th and 18th centuries represent the «golden age» of Naples for art, music and architecture, it's also true that there are many shadows as well as light, marked by contrasts between the Viceroyalty and Barons and the Church whose political, legal and economic power actually represented «states within the State». In the middle, a weak bourgeoisie and lower classes without social awareness and rights. Nonetheless, a ferment of innovations was running through the city: in 1611 the

Academy of the Oziosi was founded and in 1663, following the examples of the Academy del Cimento of Florence and the Royal Society of London, the Academy of the Investiganti, which critically revised old Aristotelian models and spread a new scientific culture. Galileo's teachings became heritage of local scientific circles and the study of natural philosophy through theory and experiments sought «hidden Truth in the Book of Nature»; absent in the European scientific field, a school of Mathematics, amongst the most advanced in Italy, was founded advocating the introduction of algebraic methods in geometry.

In the following century, with increase of population, trade and exchanges with other countries, the city became more and more cosmopolitan: destination of *grand tours* for Englishmen, Frenchmen and Germans drawn not only by the discovery of antiquities but also by the intellectual ferment, contacts with research centres abroad became intense. Debate in the Academies —for example the Royal Academy or of Medinaceli— and in private studios was lively and intense: Descartes, Newton, Leibnitz, the calculus treatises by De La Hire and de L'Hopital, the famous texts by Belidor and Borra were subject of great debate. In the elaboration of Newtonian thought, two parties formed. On the one side, those who basically accepted Newton's empirical and descriptive aspect —as is the case of *Newtonianesimo per le dame* (Algarotti 1737)— and on the other, the richer and complex one by Pietro de Martino who published *Philosophia naturalis istitutiones libri tres* in 1738.

With the conquest of Naples by Charles of Bourbon in 1734, a period of great changes finally started: the reduction of the power of barons and clergy was accompanied by reforms in the school and University system strongly upheld by Antonio Genovesi who saw in the creation of expert technicians the necessary condition for the young Kingdom to compete with other States: hence the birth of schools and military Academies where technical subjects and mathematics were the basis of studies.

It was a necessary and courageous decision in that context which however penalized scientific Academies whose best scientists were called to work in government and schools; whilst in France, England and the Netherlands Academies continued to be breeding ground of scientific culture in progressive and rapid development, in Naples they were emptied. Furthermore, economic hardships due to taking part in the War of the Austrian Succession, the plague in Calabria and finally Charles Bourbon's rise to the Spanish throne marked the start of the decline: too brief and discontinuous, the social and economic reforms were put aside and though the ideas of the Enlightenment were widely shared by scholars, they did not spread to society at large. The gap between the *élite* and the Crown grew, culminating in the Parthenopean Republic and its dramatic end: the ensuing Restoration led to the massacre or exile of good part of the intellectuals of the city while the old ruling class quickly took up power again.

In these difficult conditions, remarkable figures distinguished themselves: remaining in our field, mention must be made to Vincenzo Angiulli, lecturer at the *Reale Accademia della Nunziatella* who published in Naples the *Discorso intorno agli equilibrj* whose «action law» is the present day «principle of virtual work»; Nicola Carletti and his *Instituzioni di Architettura Civile* (1772) still influenced by 16th century treatises and Vincenzo Lamberti, whom we'll delve into soon in some depth, quoted by Bernoulli and Milizia.

Born in 1740, Vincenzo Lamberti, engineer and member of the Royal Academy of Science and Fine Arts of Naples, published his first work, *Voltimetria retta*, in 1773; two years before he had had a harsh debate with Fuga and Vanvitelli about the cause of the damages of the cupola of Gesù Nuovo in Naples. Vanvitelli, an architect with vast experience, was in favour of demolition; he had just worked in Rome on the consolidation of the cupola of St. Peter, whose project was elaborated by the mathematician Poleni; on the other side, Lamberti, younger and with less field experience but aware of new theories aiming at bringing together experience and mathematical control of the problems, was in favour of restoration. Of course, the cupola was demolished but the negative experience brought him to deepen his researches on the causes of damage, which he delved into in the last part of his *Statica degli Edifici* (Naples 1781) which represents the first original research in this field.

From the introduction, the aim of *Statica degli Edifici* is clearly set out: put at the disposal of technicians a useful instrument to calculate the structural elements, hence the language deliberately accessible to all and the rejection of «rigorous

mathematical terms». He criticized Architects for giving too much weight to Vitruvius's *venustas and concinnitas* sacrificing the *firmitas* in constructing buildings either too weak or uselessly over-dimensioned. He contested the theoretical foundations of treatises of the past: solidness and stability don't derive naturally from the respect for eurhythmy, for the module or symmetry nor are «imperfections of the materials» the cause of fractures. Bearing in mind Galilei's assumption that matter does not obey «abstract and ideal reasoning», Lamberti was convinced that only experimental data of the resistance of materials and an accurate use of mathematics ensured the stability of buildings: investigation of physical phenomena, governed by independent laws, started to move definitely away from consolidated rules repeated for centuries.

The brief space at our disposal can only allow treating what seem to us the most significant aspects concerning the statics of vault structures.

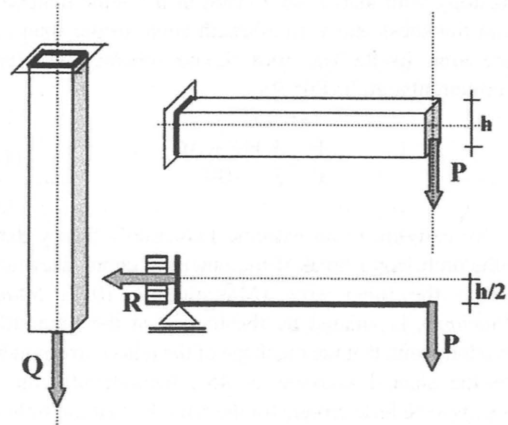

Figure 8
The angular lever

Resistance of the materials

The first aspect to underline is Lamberti's attention towards the resistance of the materials (Chapter 3, book II of his treatise). On the basis of known experiments (Mariotte, Parent, Musschenbroek, etc.), he assumed a priori that «there was no constant ratio between the *absolute force* —breaking load under traction— and the *relative force* —breaking load under bending stress— «and organised a series of experiments to determine the values of the «relative resistance» of the various materials used in Campania: tufa, piperno, lime and pozzolana. The tests were carried out on small prismatic samples with square base, fixed at one end and loaded at the free base, or resting on both ends and loaded at midpoint. The experiment data were then revised (using the equilibrium of the angular lever and the hypothesis of constant distribution of the resistance on the section of maximum stress, as Galileo had already done), in order to obtain values independent of sample geometry and of test methods (Fig. 8).

Lamberti thus obtained the following values of the breaking load of cube samples with the side of 1 Neapolitan palm, fixed at one end and loaded at the free end: Campania tufa (*rotoli* 1.873); piperno (*rotoli* 10.080); lime and pozzolana (*rotoli* 939).[2]

These elements were sufficient for him to obtain theoretically, with only the aid of geometrical similarities, the necessary formulae to dimension the beam elements under different constraint and load conditions. Particularly original and interesting are the rules concerning the determination of the ultimate load, R, of arches, semicircular and not, obtained by

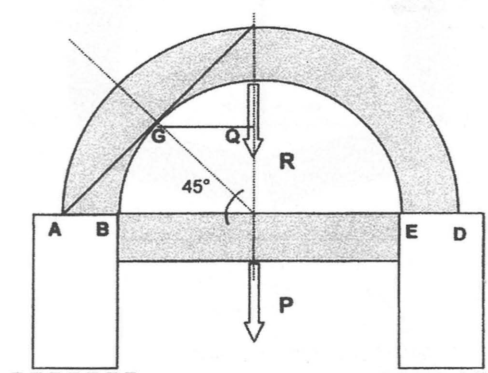

Figure 9
The angular lever

analogy with that, P, of a beam of the same material and thickness and with a length equal to the span of the arch itself. The formula he obtained for the semicircular arch (Fig. 9).

$$\frac{P}{R}=\frac{3}{5}\,\frac{Gq+AG}{BE}, \tag{1}$$

by carrying to an extreme Leonardo's theory that «the arch won't break if the outer arc chord does not touch the inner arc» (Marcolongo, 1937, *Studi Vinciani*), is vitiated by the theory, at the time still predominant, that the breakage of the reins corresponds to the slanted voussoir at 45°; theoretically, there seems to be little ground for the ratio 3:5 that multiplies the geometric data which, for Lamberti, represents the effect of the different number of fractured sections that characterize the collapse mechanisms of the two structural elements compared (Fig. 10).

The resistance of the segmental arch is obtained by a similar formula and the weakest joints are determined as shown in figure 10: it can be noticed how the collapse sections tend towards the springer sections until they coincide with them when the arch joins the beam.

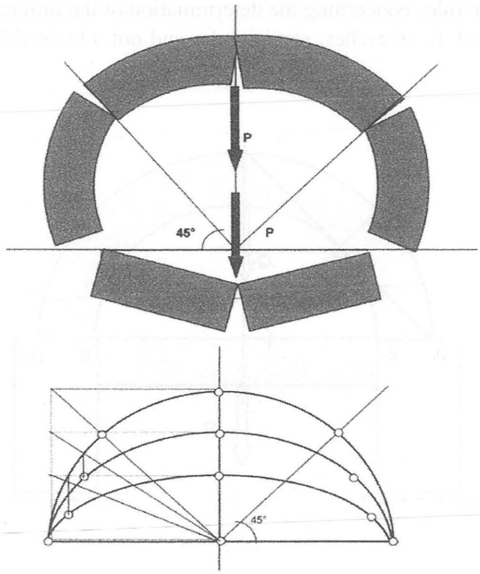

Figure 10
The collapse mechanism

Arch and vault thrusts

In the analysis of these elements, Lamberti singles two types of problems:

a. The minimum thickness to give to the vault so that it resists to its own weight and of the elements loaded on it;
b. The thickness of the piers.

In the formulation of the first type of problems, he essentially applies formula (1), or its derivatives for the segmental arch, obtaining firstly the load bearing on the arch and then the thickness of the beam of equal length to the free span.

The way he delves into the type b problems seems more interesting. The theoretical instrument is again that of the angular lever and the end formulae, worked out by long chains of geometric similarities, are in fact revisions and adaptations of the ones obtained in the analysis of isolated walls. The originality of his approach, in our opinion, is to be found in his conclusions: for each significant type of vault, the author numerically solves a standard problem; from this, he obtains factors that don't change with geometric data variations of the single problem and hence defines a new *practical* calculation procedure combining these constant terms with the variable data.

For example, to determine the thickness of the abutment of a round barrel vault, after calculating the dead force P exerted by the arch, Lamberti arrives at a first value «a» of the thickness by applying the formula previously obtained for the isolated wall. In the case examined, however, force P does not pass through the pier edge and hence the solution must be corrected to determine the new thickness, x+a, to re-establish balance (Fig. 11). Thus, the following expression is obtained

$$x=\frac{a\times m\times n}{a\times h+a\times m} \tag{2}$$

that measures the required thickness.

Applying the above procedure to a vault made of Campania tufa with a radius of r_m=8 palms and an impost height of h_m=24 palms, Lamberti obtains the dead force P_m= 23,17 *rotoli* and the thickness of the pier a_m=6,4 palms. These data are sufficient to

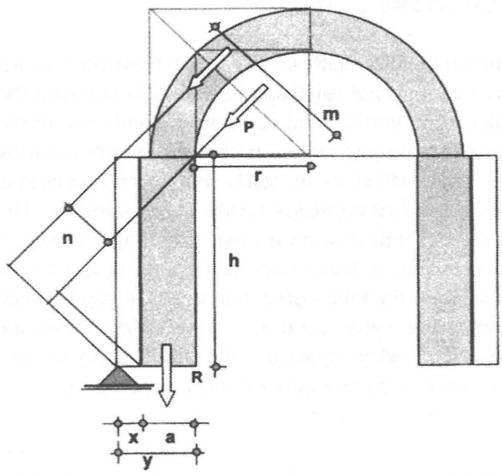

Figure 11
The thickness of the piers

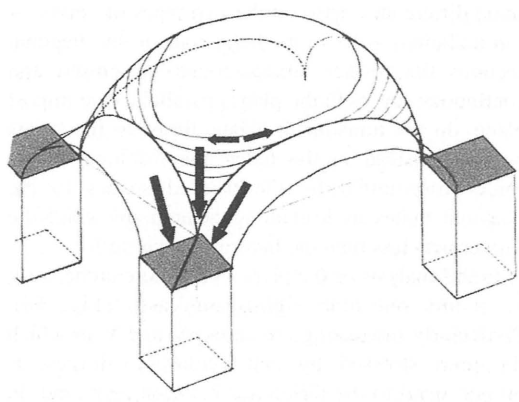

Figure 12
The cap vault's static

determine the thickness of the abutment of any round barrel vault made of Campania tufa with the formula —*pratica I*—

$$y=\sqrt{\frac{P\times r\times h_m\times a_m}{P_m\times r_m\times h}}=\sqrt{\frac{P\times r\times 24\times 6{,}4}{23{,}17\times 8\times h}}$$

where r, h and P stand for the radius, the impost height and the dead force.

Similarly, he arrives at six other *praticas*, founded on six basic numerical solutions covering the different types of barrel vaults, with or without piers exceeding the imposts, and flat arches.

The static behaviour of the cap and cross vaults is deduced from the wall bond and the proportionment of the system refers to the most stressed basic components. In cap vaults, isolated, on piers, Lamberti indicates as elements to be «calculated» the perimetrical arches and the piers (Fig. 12). In the former, he observed that the acting forces, conveyed by the cap vault calotte increase from the middle to the impost, but must not be considered in their entirety since the orthogonal component at arch level «is destroyed» in meeting the «obstacle of the thickness of the arch». To size the abutments it's necessary to consider the thrusts coming from the big arches and the action of the vault pertaining to the pier. Their resultant will in any case have the direction of the diagonal of the space to be covered and consequently the pier section will need to be an homothetic figure to it, of which he determines the diagonal length. In the cross vaults, created by the intersection of two semi-cylinders or by two semi-spheroids (cross vaults with or without «reguglio», i.e. higher keystone), the most stressed elements are the same as in the cap vaults and therefore the author reaches the same operative conclusions (Fig. 13). The

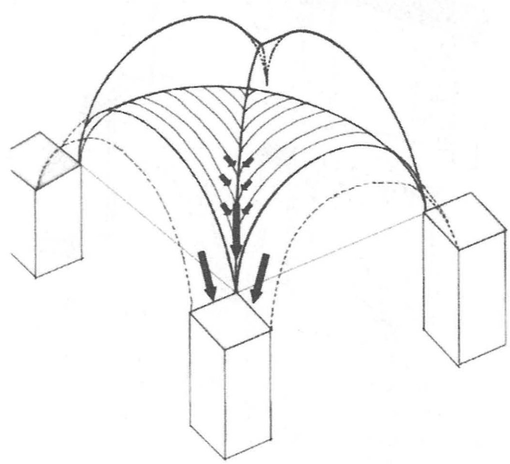

Figure 13
The cross vault's static

static differences between the two types of vaults —cap and cross— lie in the behaviour of the diagonal section: the former, characterized by closed and continuous curves in the planes parallel to the impost plane, do not transmit the plane thrust to the layers beneath, instead in the latter, the resultant of the forces transmitted by the vaulted spaces to the diagonal arches are lead along the arch «by which the former acts less than the latter in the corners».

In the analysis of the piers where more arches rest, he points out nine significant cases (Fig. 14). Particularly interesting are cases IV and V in which the pier, stressed by two «contrary» forces, is proportioned to the difference between them and, in extreme cases, is arbitrarily small if the two forces are equal. If, instead, three forces act on the pier, two «contrary» and the third «converging» (cases VI and VII) or four thrusts «converging» in pairs (cases VII and IX), the problem is similar to the one with two «converging» actions, replacing the two «contrary» forces with their difference. In particular, if one of these is nil —equal and contrary forces— the size of the pillar is determined in the direction associated to the «converging» force whilst the other is discretional.

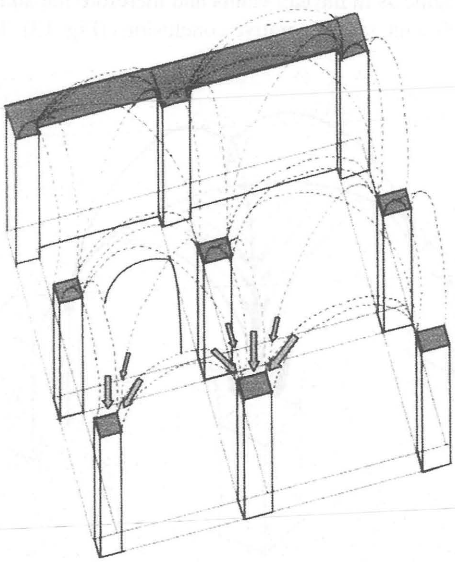

Figure 14
The static of the piers where more arches rest

Conclusions

The stimulating ideas emerging from the treatise we've tried to illustrate briefly have led us to compare the rules of proportionment laid out by Lambert with the geometry of some buildings; the small number of tests has not enabled us to reach significant conclusions though the first numerical results are encouraging. The next steps will concern a more widespread survey to check if the deduced rules correspond in fact to the practice of the time, and a control of the gap between Lamberti's values and the values that would be obtained today applying the limit analysis and correcting some blatantly mistaken hypotheses.

Notes

1. DE MAGISTRIS ARTIUM SEU ARTIFICIBUS TITULUS LXXXII PRAGMATICA PRIMA *in Blasaius Altimarus,* Pragmaticae, Edicta, Regiaque Sanctiones Regni Neapolitani (Naples, 1682)
2. 1 Neapolitan palm or span = 26,3 cm; 1 rotolo = 0,891 kg.

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Detailed references on treatises, scientific knowledge and theories of vaulted structures can be found in Benvenuto (1991) and Di Pasquale (1996).

The use of adobe in the traditional buildings of Sardinia Typological and construction innovation between XIXth and XXth century

Maddalena Achenza

Earthen architectures, or architectures in which earth as a building material is the main component, are nowadays frequent topic of studies and researches run world-wide. Representing the archaic local building tradition they witness the specificity of the technical solutions adopted in different cultures, climates, habitats.

The island of Sardinia is retained in Italy the Region with the most conspicuous earthen built heritage, and therefore represents, at least in a national context, a privileged reference. The use of earth as a building material is concentrated in the southern part of the island in an area included roughly between Oristano (north) and Cagliari (south) along the plain called Campidano, where the diffusion was capillary and used to cover in a close past almost the totality of the buildings in rural and town settlements. Except for the capital, built with the limestone extracted from the hills on which the town was founded, earth was used with excellent skills to build houses as far as public buildings; the typological variety shows itself in the elementary residential cell as far as in palaces, in shelters for the animals as far as in industrial structures. Nevertheless the earth building is mostly represented in general by a court house whose repetition creates the morphology of our settlements. An approximate estimate lists about 90.000 buildings spread in almost 30 towns, which means that about a third of the Sardinian traditional heritage is built with earth. The used building element is adobe, formed generally in the standard size $40 \times 20 \times 10$ cm, but changing measures according to specific local uses and personal needs. (Figure 1)

The area in object is an alluvial plain, where woods and stones were of more difficult reach. The soil available in the plain of Campidano shows often a very different composition in the various areas, changing the contain of minerals and clay, as much as granulometry and texture. The nature of the clay though is almost everywhere adequate to the specific purpose of the adobe production, except rare cases the

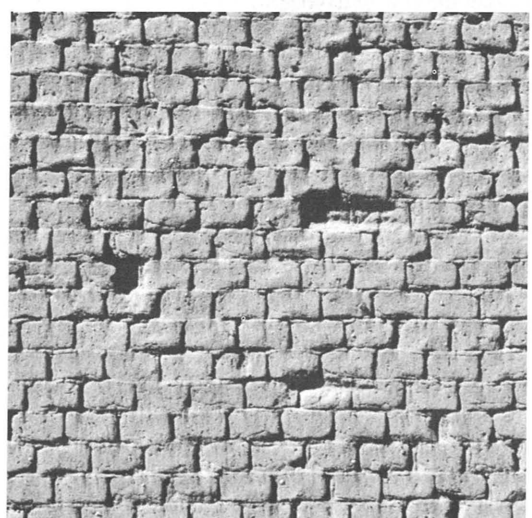


Figure 1

physical characteristics fall in the range of the adapted soils, so that it was generally possible for the private builder to use the materials excavated *in loco* for the foundations to make his own bricks (Angius and Casalis 1833).

Several interviews made in our early researches to old producers and builders testify the spread culture of self building, the diffusion of deep-rooted skills possessed by everyone, able to compose optimised mixtures of earth, straw and water, to obtain a product of acceptable resistance. It is still readable in the composition of the ancient bricks the efforts the producers had to make to bring the mixture of a bad soil to an acceptable building «standard»: for example, a bigger amount of straw testify the excess of clay content.

In the bigger centres it was also possible to find real production plants[1] where one or more men were able to produce manually 200–300 bricks a day. The production system was the same found throughout the world, mainly manual, rarely helped by animals. The mixture was prepared one day in advance with earth, water and straw. On the next day, using wooden forms the men produced the bricks laying them directly on the ground and letting them dry in the sun.

This production stood alive in most cases up to the end of the 50's, when the same producers simply replaced earth with cement, easier to work and apparently more durable, and introduced into the market the concrete block.

These years stated the end of a building culture, the interruption of traditions and *savoir-faire* carried over since thousands of years, as it is demonstrated also by recent archaeological founding.

As mentioned before, the most frequent built typology is represented by the court house. (Figures 2, 3)

In order to better identify this buildings we can still use the description made in 1941 by the geographer M. Le Lannou (Le Lannou 1941):

Figure 2

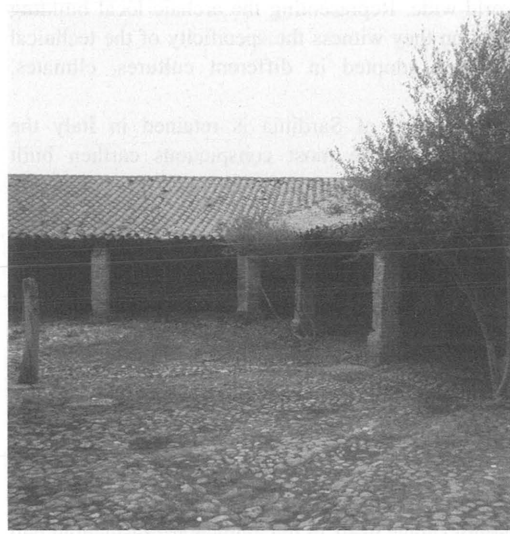


Figure 3

> The house of the south is the most complex and at the same time the most complete of all Sardinian rural buildings. The house opens on the road with a wide portal often rectangular, more frequently surmounted by a camber or round arch . . . The portal is the only entrance to the house. It opens to a square courtyard, completely surrounded by buildings and walls. The house itself stands on the opposite side of the portal. It is preceded by a porch with a descending roof kept by wooden or brick pillars . . . Along one or more sides of the yard we can find also some more perfunctory buildings, sometimes open on the courtyard, too: shelters for the cattle, the cart and the tools, the wood supply, the oven, the mill and the donkey that turns it. When these buildings lean on the wall towards the road then the portal becomes access to a tunnel like to a monumental entrance. In the courtyard a

well is never missing, often in open air. The porch (*lolla*, in local dialect) is more than an external element, a real living room . . . In the windy and hot plains of southern Sardinia, the loggia is a useful remedy to climate excesses. Windows and doors of the house open to the court giving light and air to the rooms . . . [2]

The court is then reduced to essential, structurally introverted, communicating with the external just through the entrance portal (unless later divisions or obstructions do not force to open new doors to the road). This is also the reason why the sequence of tall walls along the roads, scanned by portals, is the representative sight of southern Sardinian town centres. (Figure 4)

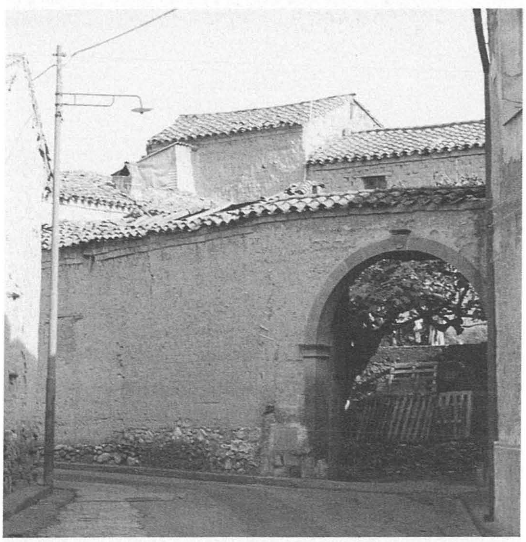


Figure 4

Despite this homogeneity, the typology of the court house hides an extraordinary variety of differences, given by the multiplicity of the building specific elements; dimension and extension, orientation, access and relation to the road, internal relationship among the buildings, can be combined to a wide-ranging record of cases. For an easy interpretation we can resume the diversities depending on three main characters: social typology of the owner (poor, well-to-do, rich) and consequent size of the property, typology of the relation through the access to the road, typology of the buildings. (Figure 5)

DIMENSIONAL BUILDING CLASSES DEPENDING ON OWNER SOCIAL STATUS	TIPOLOGY OF THE RELATION TO THE ACCESS	BUILDING TYPOLOGY (RESIDENTIAL OR INSTRUMENTAL)
great court of rich owner	south entrance	single level
middle/small court of well-to-do owner	north entrance	with partial raising
minimal court of poor owner	lateral entrance	with loggia
	corner entrance	simple body of the building
		double body of the building
		extroverted (variant *a palattu* - palace)

Figure 5

This schematisation suggests the variety of the possible cases that lead to a non-static texture. Actually it is the former scarcity of the buildings in the urban texture given principally by the presence of the court, the prevalence of empty on full, the main cause allowing intense modifications.

The process of division due to inheritance sharing became later the prevalent reason of morphological modification, and as a result, of urban transformation: the first visible consequence was the raising of a wall in the middle of the court. The filling, at least partial, of the empty space of the court followed immediately after. The new disposition of the spaces forced new openings of doors and windows, often the closure of the loggia; the buildings grew in height and length coming closer up to touch each other. It is almost always easy to read these morphological transformations confronting the cadastral maps of the end of 1800–beginning 1900, although sometimes the interventions have been so deep and important that only with a radical work we are able to recognise now the original situations. (Figure 6)

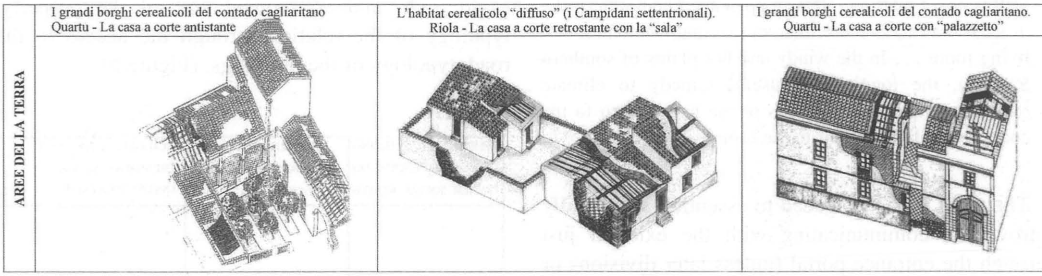


Figure 6

The adobe shell

If the morphology of the settlement system was often changing, according to the succession of owners and new needs, the building techniques used throughout the centuries remained the same, showing a well established possess of the construction skill.

From direct witness (now disappearing as the former builder and adobe producers have reached a certain age) and from rare specific documents we were able to find about a few of our public buildings we got the information on the provenience of some of the building materials, as far as about the wall construction, the horizontal closures, the openings and their frames, the finishing, the plasters and mortars, the particular details of more difficult building parts.

The building laid rarely directly on the floor, without foundation. More often some excavation occurred from which the soil was then used to produce the adobe bricks necessary for the raise of the walls. These excavations were then filled with non worked river stones, bounded with clay or lime mortar. The stone foundation was frequently carried 50–80 cm up from the ground, to keep the adobe wall away from possible humidity raising from the soil for capillarity. (Figure 7)

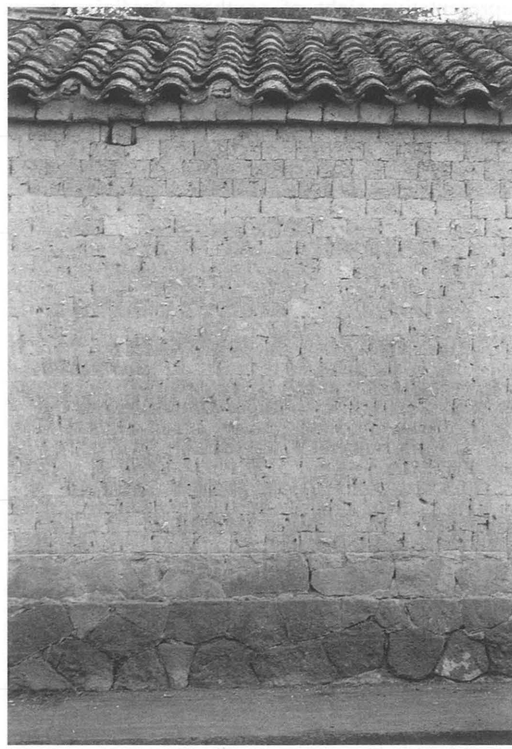


Figure 7

The walls were completely build with adobe, in the frequent size 40 × 20 × 10 cm (this measure varied in the size but never in the relation 4:2:1) produced directly in the building site, more rarely bought. The lay of the wall was whole-brick, and previewed the pose of the bricks always by the width with offset joints. This disposition was kept also in the corner and in the T-joints, where the bricks were cut at? of the length to avoid the superposing of the joints of the next layers. In more rare cases the bricks were laid alternatively by the length and by the width, in a double brick thickness, so that the corner could be realised with entire bricks (Sanna 1999) (Figure 8)

The internal walls were sometimes made with the same adobes put by the width, the wall having a thickness of 20 cm.

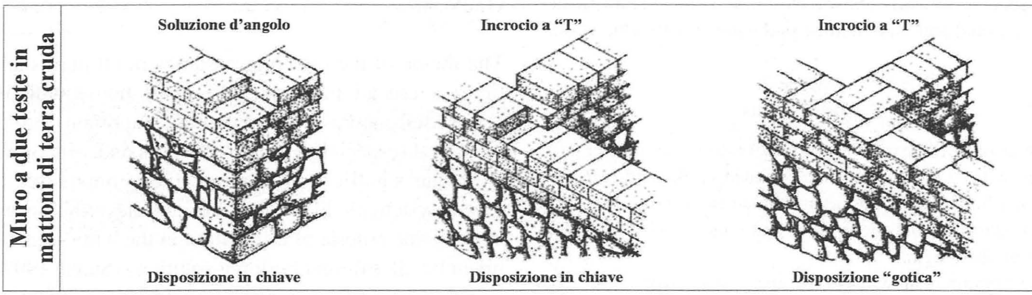


Figure 8

The mortar employed was traditionally made of the same mixture used for the fabrication of the bricks: clay, sand and water; just the straw was here not any more added as the shrinkage effects (frequent in soils rich of clay) were less important in thin layers. At the end of the 19th century, when a significant innovation appeared to change the traditional way of building, the mortars would systematically preview a mix of clay and lime in a relation 2:1 for all type of walls, including the adobe ones.

The plasters were traditionally made in at least two layers: the first one creating the right support for the finish was again of a mixture close to the one used for the bricks, with a finer soil, often sieved and with addition of straw. The straw was often replaced with grain chaff, very short fibres that allowed an easier workability. The plaster was laid in a thickness of 1–1,5 cm. The final finish was always made with a lime plaster, at least on the exterior walls where a major protection was needed. An intermediate layer was also often realised, with less straw and a small percentage of lime.

One of the most important documents that gave us an extraordinary amount of information on traditional building processes and of intervention on existing structures is represented by the Capitolato d'Appalto that accompanies a project for the enlargement of the house belonged by Antonio Nobilioni in Quartu Sant'Elena. The house was bought by the Municipality on 3rd august 1853, with the intention of moving its head office after the renovation of the building. (Figure 9) In 1868 the planner addressed to the Town Council a report explaining aims and methods of the project, all completed with the mentioned technical

Figure 9

specifications which define exactly materials and building characters to be used in the work:

Article 21
Brick walls – 1° burnt bricks

All arches of the porch will be built with this bricks, together with those of the hall and the staircase, bound with a lime and sand mortar taking care that this last one will have to be finer and carefully sieved before being used.

2° adobe bricks

The adobe wall will be cemented with a well prepared clay and sand mortar. The brick course will be with alternate joints, namely the thickness of each course and its length will be formed by bricks disposed by the length and by the width, alternate, so that the joints would never correspond not inside and not outside.

In general all walls will be plastered, rendered and floated with a sand and lime mixture and painted with whitewash.

Article 22
Brick vaults

The porch will be covered with cross vaults with lower rises; they will be built with the mentioned bricks and cemented with a lime and sand mortar, painted twice up to 2/3 of the arch starting from the impost, and closed with a key brick put by the length.
The entrepreneur will choose the cast he will retain most adapted, provided that it is solid and well built, the Administration will adopt it, too.[3]

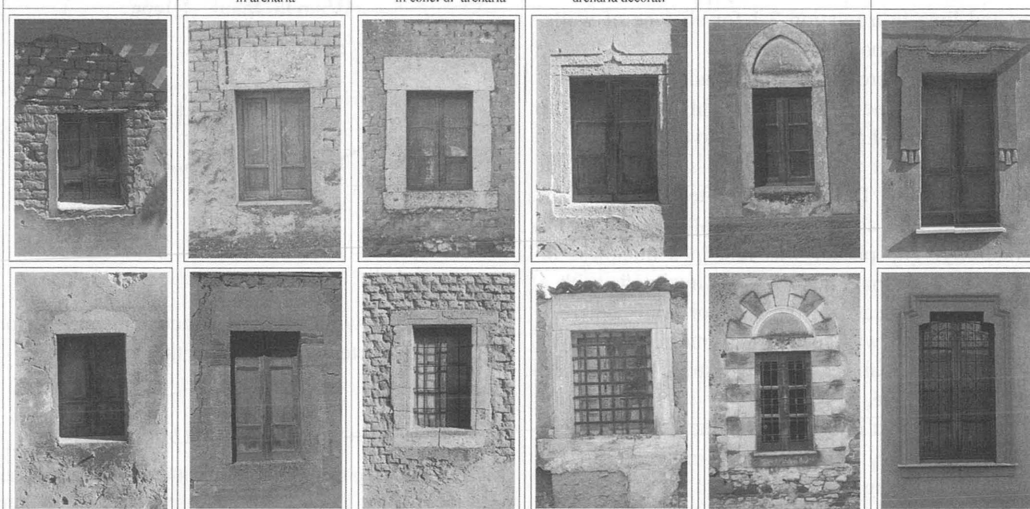
It is also to be noted that these indications represent an exception among the great amount of the buildings that continued to be raised in a quite archaic manner.

Openings

The theme of the openings assumes in all places and times a crucial meaning. The court house with its introverted nature gives in a more important way to the portal (especially) and to all doors and windows a particular significance. It is obviously impossible to give a systematic list of all cases, but nevertheless we chose some criteria of orientation in the wide scenario given by all different building solutions (Sanna 1999).

The first case is represented by the elementary window, a simple opening obtained in the adobe wall kept by a wooden (juniper) architrave, lacking in lateral supports. This type of window could have a very small dimension, not exceeding the square meter. In rare cases the architrave could be replaced by a flat or depressed adobe arch.

A) Architrave ligneo	B) Architrave e/o stipiti in conci di arenaria			C) Architrave sormontato da arco	D) Finestre con forme particolari
	B1) Architrave monolitico in arenaria	B2) Architrave e stipiti in conci di arenaria	B3) Cornice in conci di arenaria decorati		



L'architrave è realizzato con un elemento ligneo, solitamente di ginepro, che si appoggia su stipiti in mattoni crudi o cotti o, più raramente, in conci squadrati in arenaria.	L'architrave è realizzato con un monolito in arenaria o, in alcuni casi, in basalto che si appoggia su stipiti in mattoni crudi o cotti. La scarsa capacità di resistere agli sforzi di trazione del materiale lapideo impone all'architrave generose dimensioni.	La struttura dell'apertura è ottenuta con un sistema trilitico di conci squadrati in arenaria. I piedritti si posano sul davanzale anch'esso costituito da un blocco lapideo.	Non di rado, i conci costituenti la struttura dell'apertura sono dotati di elementi decorativi, fra cui ricorre quello della "fiamma" di origine catalano-aragonese.	Nelle case più ricche la rigida geometria del trilite viene addolcita con un arco sormontante l'architrave, realizzato in mattoni cotti o in conci di arenaria. Oltre alla funzione decorativa, ad esso è assegnata quella strutturale di elemento di scarico.	Gli edifici più signorili, alcune volte, sono arricchiti con un sistema di aperture dalle forme del tutto particolari.

Abaco delle finestre di Riola

elaborazione: Carlo Atzeni

Figure 10

A development of the previous example came due to a first contamination with the more refined building art of the late 1800. In this case the architrave was surmounted by a discharging arch, made with burnt bricks, and the lateral supports were also brick made well joined to the adobe wall. This type of opening had at least two shutters and width dimension above 1 meter. The use of burnt bricks underlines the better cure put in details like the edges of the window side supports, well more resistant than the adobe ones.

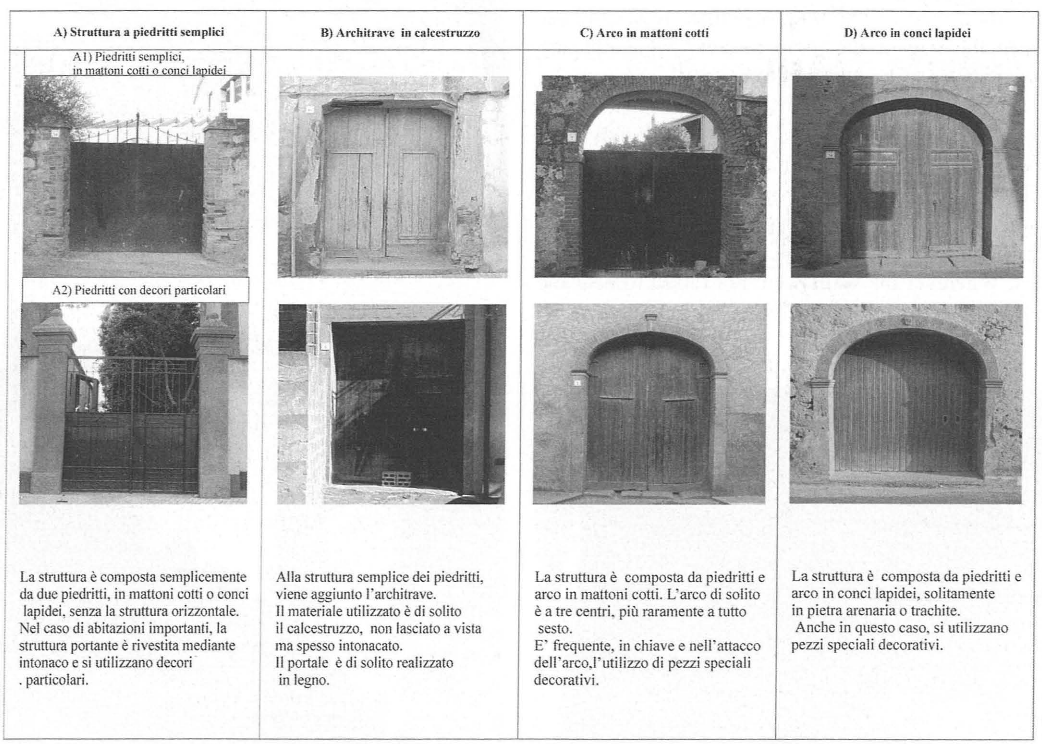
The case of the opening realised with an arch is definitely to be referred to the later *typology «a palazzu»*. It is characterised by a camber arch all made with burnt bricks, often exposed face. The positioning of the bricks is sometimes rotated to give the proper angle to the internal jamb, and confers to the external the light and shade effect that characterises nowadays the historical town centres.

The last example of opening finds an external decoration made with pre-fabricated terracotta profiles, later made of cement. This type of window is obviously more appropriate to the palace typology, but it is to be found also in court houses. (Figure 10)

The entrance doors fit to the same building language, but they were not related to the court house typology and actually belonged just to the building opened on the road front. The door carried the same characters of the windows, showing the only difference in the use of stone, often worked, for the arch and the supports.

The portal had a special meaning in the court house. It was often the only entrance to the house and brought therefore to the external all information related to the owner family. (Figure 11)

The portals left nowadays are almost always surmounted by an arch, often three centred, made out of adobe or fired bricks, very rarely of stone. It is



Abaco dei portali di Villamassargia — elaborazione: Gianpietro Scanu

Figure 11

most probable though that the ancient portals were surmounted instead by an architrave, as the wooden frames still show. But if the materials, less the form, were changing, the geometrical relations were frequently constant: 2,4 m in the width corresponded to as many in height, up to the arch impost. The keystone carried often the initials of the owner or the symbol of his job.

Floors and roofs

The floors connote a constant building pattern common to different areas and related to buildings of different typology. The room type is rarely wider than 5 metres, always covered with a wooden beam structure on which a plank floor is laid. The wood beams are posed at a general distance of 80 cm and the plank floor is directly nailed on them. Very rarely is the double frame with bearing beam to be found, so the charges are divided in fact just on the two walls. The wood used was preferably juniper, replaced later (when the size of the juniper trunks were not any more sufficient) by chestnut-tree wood.

The roofs, according to the simple shape to be covered, is almost always a pitched roof with thick bearing beams (ridge board and binding rafters) laid directly on the walls carrying a secondary wooden frame. (Figure 12) A reed thatch was fixed on top, on which an earthen screed was keeping the channel tiles. Wherever the walls were not raised to bear the roof weight, a system of trusses was realised. This simple system was very often used as the span to be covered were normally contained. For bigger spans stone or brick pillars inserted in the adobe wall carried the point loads of the truss.

The eaves are a very important building detail, serving to crown the building and at the same time protect the walls from rain— and stream waters. The solutions found are various and employ different materials, starting with the simple overhang accomplished with channel tiles disposed so that they

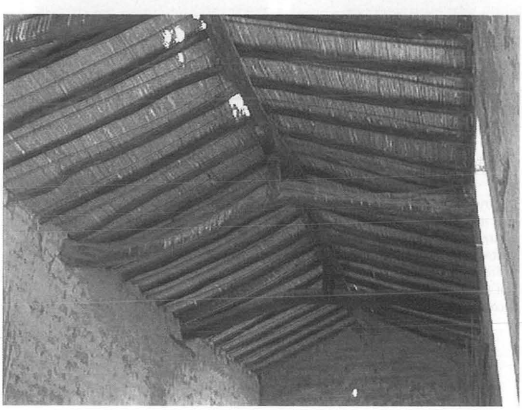
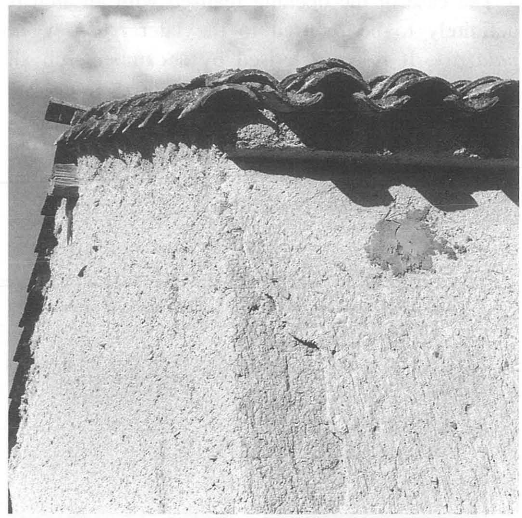
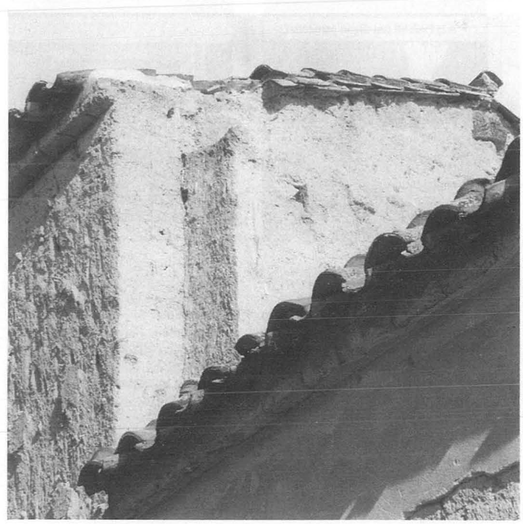


Figure 12





Figures 13a and 13b

hang over the wall. It is a common building art spread in all Mediterranean areas, also shared with stone wall typologies. (Figures 13a, b)

Another example is related to the channel tiles laid on a overhang produced with more tiles, in more refined cases replaced by burnt bricks. This last solution would be more frequent during the course of 19th century when the new town fashions started to influence the building art of the countryside. At the end of the century, according to new norms, the parallel gutter started to be mandatory: the first adopted immediate solution was to place by the length a line of tiles on top of the wall: the tiles would be superimposed and well sealed in order to avoid any water infiltration in the wall below. From this the consequent evolution was the realisation of the crown wall at whose inside the gutter could be hidden.

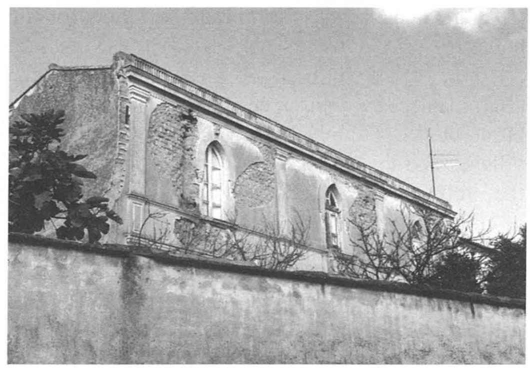
In the domain of the research carried in the past years at our Department[4] we synthesised most of these topics in various publications included a Cd-rom titled «Arte del costruire-Guida al recupero dell'edilizia storica di Quartu», processed in accordance with the recent studies that deal with the knowledge of pre-modern and modern building cultures and techniques with the aim of restoration and rehabilitation. We referred especially to the stream of the Manuals for the Rehabilitation of Rome, Città di Castello (Giovanetti 1992) and Palermo where this type of approach is evident. The Codex of Practice written more recently for the rehabilitation of the «Sassi di Matera» by Antonino Giuffrè (Giuffrè 1997) has improved this studies adding new contributes about the operational systems. Manuals and Codex of Practice are giving now an original and precious contribution for a contemporary understanding of the various building practices bringing us to a revaluation of the culture of materials in order to solve in an appropriate manner the problems joined to the knowledge and the intervention on historical centres and traditional architecture. From this point of view our research defined in depth each building element, its production, the put into realisation, the durability, its relation to the context.

Typological and construction innovation

At the end of 1800 and during the course of 1900 a major change occurs: new typologies come close to the basic type of the court house and integrate in the urban context, responding to a demand of new functional needs and especially a new necessity of self social-representation of the raising rural bourgeoisie. (Figure 14)

The most evident effect is represented by the introduction of the new typology of the front road palace inserted in an urban texture of court houses. The introversion of the house gets completely upset in order to gain a face on the public road where new decorated façades show openings and balconies more adapted to a new lifestyle.

Evidently this new model gets perfectly inserted in between the ancient tissue, and lives together with the adobe introverted houses, never in conflict. The



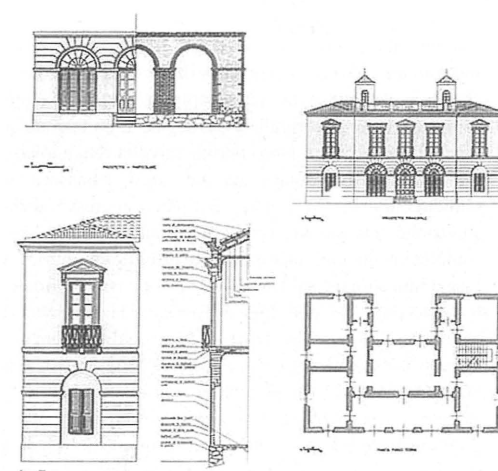


Figure 14

common building material is used as always, the differences being in the reinforcements of the corners, posts, architrave and cornices, and the importation from the town of new building materials (especially iron) for elements like balconies and balusters.

Among the residential typologies of new conception of the end of the 19th century some rare cases are also to be mentioned that concern adobe buildings which not only occupy the court, but in fact overturn its constructive logic. It is the case of some villas replacing the open spaces of the town centres: one particular example is to be found in Quartu Sant'Elena and it is represented by the Villa Fadda. It is a great neo-classic complex built on the influence of the famous school of Gaetano Cima in Cagliari. Of the same architect another villa in the same style was built in San Sperate.

Another mention is to be made about public buildings and industrial settlements. The use of adobe is to be found in both of them, although often accompanied by reinforcements made of stone or burnt bricks. Examples are to be found in many villages and towns, such as Quartu, Selargius, Pirri, the mining area of Sulcis where townhalls, distilleries, slaughter-hauses, glass-work plants, brick-kilns all built mainly with adobe bricks, fusing tradition and innovation, ancient and actual skills, and influencing the following building production.

Notes

1. Among others, Quartu Sant'Elena-areas *Pizz'e Serra* and *Funtan'e Ortus*, Selargius, Villasor-area cemetery
2. «La casa del sud é la più complessa e anche la più completa delle case rurali di Sardegna. L'abitazione si apre sulla via con un largo portale talvolta rettangolare, più spesso sormontato da un arco ribassato o semicircolare ... Il portale é l'unico ingresso della abitazione. Dà su un cortile di forma quadrangolare, completamente circondato da costruzioni o da muri. La casa d'abitazione è sul lato del cortile opposto al portale. E' preceduta da una loggia formata da un tetto a spiovente sostenuto da pilastri di legno o di mattoni ... Su uno o più lati del cortile ci sono delle costruzioni sommarie, spesso aperte anch'esse sul cortile come la loggia: sono i ripari per i buoi da lavoro, per il carro e gli utensili, per la provvista di legna, per il forno, per il mulino rustico e per l'asino che lo fa girare. Quando questi ambienti s'addossano al muro che bordeggia il cortile dalla parte della strada, il portale dà accesso ad una specie di tunnel che forma come un'entrata monumentale. Nel cortile non manca mai il pozzo, spesso all'aperto. La loggia (in sardo sa lolla) è, più che una galleria esterna, una vera e propria stanza d'abitazione ... Nelle pianure surriscaldate e ventose della Sardegna meridionale, la loggia è così un utile rimedio agli eccessi del clima. Sulla loggia si aprono le porte e le finestre dell'abitazione propriamente detta. Le stanze interne prendono luce solo di qua, e tutte danno sulla loggia, con porte a vetri o con una finestra».
3. Articolo 21
   Murature Laterizi – 1° Mattoni cotti
   Saranno costrutti con mattoni così detti del campione tutti gli archi del portico, del vestibolo, ed i rampanti e ripiani della scala, cementati con malta di calce e sabbia avvertendo che quest'ultima dovrà essere alquanto più fina e prima di esser posta in opera diligentemente crivellata.
   2° Laterizi o mattoni crudi
   La muratura in mattoni crudi, sarà cementata con impasto di terra e calce ben manipolata.
   I corsi dei laterizi saranno a giunti alternati, cioè lo spessore di ogni corso e la sua lunghezza sarà formata da mattoni disposti in lungo ed in traverso, alternati in modo che le commessure di un corso non corrispondano a quelle del corso successivo nè internamente nè esternamente.
   In generale tutti i muri saranno intonacati, arricciati e frattazzati con malta di calce e sabbia ed imbianchiti con latte di calce.
   Articolo 22
   Volte in mattoni
   Le volte che dovranno coprire il portico saranno a crociera ed a monta alquanto depressa, verranno costrutte con mattoni del campione cementati con impasto di calce e sabbia, e imbiancate in doppio fino ai due terzi dell'arco, a partire dall'imposta e chiuse in chiave con un mattone di punta.
   Per l'armatura l'impresaro potrà scegliere quel sistema che stimerà migliore, purchè sia solido e ben congegnato, verrà adottato anche dall'Amministrazione.
4. «*Materiali per un Manuale-Atlante per il recupero dell'insediamento e del paesaggio rurale della Sardegna*» (Scientific coordinator: Antonello Sanna-Università di Cagliari), integrating the Ricerca Scientifica di Rilevante Interesse Nazionale Tradizioni del Costruire nel Territorio Nazionale: «*Continuitá ed evoluzione delle tecniche edilizie per la salvaguardia ambientale del contesto insediativo minore*» (Scientific coordinator: Adolfo Cesare Dell'Acqua-Università di Bologna).

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Among the general texts we recall some other volumes:

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Galdieri, Eugenio. 1982. *Le meraviglie dell'architettura in terra cruda*, Bari: Laterza.

Sanna, Antonello e Angioni, Giulio.1988. *L'architettura popolare in Italia. Sardegna*, Bari: Laterza.

A. Sanna et al, 1993. «Architetture in terra: tipologia tecnologia, progetto», Cagliari: CUEC.

Scudo, Gianni et al, 1997. *Le regioni dell'architettura in terra: culture e tecniche delle costruzioni in terra in Italia*, Rimini: Maggioli editore.

Sanna, Antonello-Mura, Gianni.1998. *Paesi e città della Sardegna. I paesi*, Cagliari: Cuec.

Inventing a history for structural engineering design

Bill Addis

It is often convenient to assume that history is a series of facts waiting to be discovered. But there are so many facts. The facts that we do discover are, to a large extent, auto-selected according to who we are and why we are looking for them. This influences where we look for facts and what we do with them when we have found them. The history that is so generated can be very personal, even subjective, and the great challenge is to persuade others of the value or reward to be gained from adopting a certain approach to history.

As a practising design engineer, the author's own journey into the past began with questions about how people in former times might have designed things. The search uncovered a wonderful range of material in various different strands of structural engineering history:

— the development of different types of structure (e. g. suspension bridges, timber roof trusses, dams, masonry domes);
— developments and inventions in engineering technology (especially materials, plant and machinery, and methods of manufacture or construction);
— biographies of engineers;
— the stories of individual projects;
— developments in engineering science and mathematics («theory» and techniques of analysis), and
— various non-technical issues (economic, commercial, contractual, management, social, political, etc.).[1]

Despite much searching, however, it became clear that very few people had attempted to answer the author's most burning question —how were the structures designed? To a large extent, the challenge became to create or invent the idea of a history for engineering design. Rather than using someone else's map to navigate the territory, the challenge was to draw a new map. And an important aspect of this challenge was to distil the very essence of engineering design, and, especially, to distinguish it from general ideas of «technology» and «engineering science».[2]

Engineers themselves have seldom been effective is promoting the nature of their art —they have generally been too busy getting on with projects— and this is nearly as true today as five hundred or a thousand years ago. Professor Fritz Leonhardt, from Stuttgart, was one of the most eminent design engineers of the 20th century. In a book about what engineers do he wrote:

> Der Beruf des Bauingenieurs ist in der Öffentlichkeit fast nicht bekannt. Wenn vom bauen die Rede is, dann denkt man die meisten an den Architekten. Wenn ein Bauingenieur Bauten entwirft, dann wird er in den Zeitungen als Architekt besprochen. So ging es dem Vervasser fast sein ganzes Leben hindurch. Ich bin der Meining, daß die Bauingenieure im wesentlichen selbst daran schuld sind[3] (Leonhardt, 1981).

In essence, to design an artefact is to plan what you are intending to do before you start, and to devise a practical and economic way of constructing it. With characteristic simplicity, Ove Arup once wrote that:

> Design is nothing else than indicating a sensible way of building. It includes all drawings, specifications, descriptions and detailed instructions about what should be built and how it should be built (Arup 1985).

Even definitions such as this, however, overlook a most important aspect of engineering design —the process of achieving the necessary confidence to begin building. It is one thing to sketch a magnificent bridge or roof for a building, quite another to be confident that it can be constructed and will safely carry the loads to which it may be subjected. This aspect distinguishes fundamentally the work of those engaged in civil and structural engineering from those engaged in the design of small machines or consumer artefacts, who never need to address questions as to whether their creation will work first time, or be safe, or be possible to make.

Engineering design is a process with two key outputs:

— a *description* of what has to be manufactured and built: materials, specifications, dimensions, construction method. This may be a verbal instruction, but, especially when the building is large or complex, would usually be written down or in the form of a drawing or a model; and

— a *justification* or *explanation* of the design proposals that have been made. This may be achieved on the strength of precedent, full-scale tests, experiments on small physical models, various calculations, or the use of mathematical models, nowadays based on engineering science.

A study of what engineers do can make the process seem all very mechanical, with resulting artefact an almost inevitable result. But this is not how it seems to the design engineer, him or herself. It is important to consider the challenge and the unknown aspects of a project that the design team faces. From an engineer's point of view, the principal achievement in constructing the Pantheon, or a large cathedral or the Eiffel Tower, was that some people were able to convince themselves that the resulting structure could, indeed, be built, and then that they were able to persuade the clients and even the public that they would be possible. And furthermore, they were so successful in putting their case that they were able to raise the considerable sums of money necessary to undertake the projects.

If the engineer's job is to be summed up in one phrase, then, it can be *to create the confidence to start building*. This provides a focus for historical study. How, at different times and places, have engineers found the necessary confidence to build?

If, at any time in history, the project engineer did not feel confident that a certain structure would work, he would have to decide what he needed to do in order to gain that confidence. This is the art of the engineer. Seeing and understanding how something similar has already been done is probably the most important help an engineer can get. For unprecedented or especially challenging problems, scientists and mathematicians (often called «philosophers» in ancient times) have also been an important source of expertise that can raise the engineer's confidence.

This confidence must overcome any doubts about success the engineer may harbour, as well, of course, as any doubts by the client whose money and reputation are also at risk. A history of engineering design, then, must address these different aspects of the design process:

— what was the final design for the artefact or structure?
— how was the final design arrived at or devised?
— what means did the design engineer use to increase his confidence, *before commencing construction*, that the proposed structure would work and could be constructed, sufficiently for him to begin building.

A history of confidence among engineering designers would be a challenging subject to undertake since confidence depends very much on the experience and the cultural and intellectual background of the engineer as well as the context in which the engineer is working. There are also important questions about how different engineers in the past might have perceived the gap between what they knew had already been done and what they

might have believed to be possible, though not yet know by what means. And the confidence is needed at even the most fundamental levels such as the reliability and sophistication of mathematical calculations and the accuracy of dimensions generated by geometric techniques of producing drawings of complex shapes or connections in a building. A history of engineering design needs to address these aspects of the skill. This was a challenge that faced the author when working in the team devising an exhibition about the work of engineers some years ago[4] —had do you put on display? On that occasion the answer to this question included a collection of the engineer's aids to calculation —tables of logarithms, slide rules and mechanical and electrical calculators. There were also some design engineers' notebooks with their little sketches and the calculations they made when exploring a new idea. These things really do capture something of what engineers do, but their apparent insignificance may still give only a hint at how a final grand scheme may have come about.

There are, of course, problems in using modern words like «engineer», «design» and «designer» to refer to the construction of a Gothic cathedral or, indeed, any edifice completed before the twentieth century when these words acquired their modern meanings. The author has not found it difficult to use such words to describe the past, without implying the professional demarcations, knowledge and working methods of the twentieth century. At the risk of being accused of tautology, it is possible to use them to embrace the work that must have been undertaken to complete a cathedral, without specifying precisely what it was or who did it (since we do not know). It is possible to discuss the process of designing a cathedral, for instance, without becoming entangled in questions as to whether it was designed by an architect or an engineer, or whether its structure was designed separately from the architecture. These questions are, literally, meaningless because to use the very words «engineer» or «structure» is anachronistic.

What happened, over a period of many centuries, was that the activity of designing a large building, whoever did it, was gradually broken down into more and more distinct issues. Thus the methods of designing a Greek temple or a cathedral embraced visual appearance, sense of space, function, materials and structure, as well as what we now call the internal environment (lighting, heating, ventilation and acoustics) in a single, holistic design process. The visual appearance, sense of space and function (the «architecture» in the modern, narrow sense) became a distinct concern during the fifteenth and sixteenth centuries. About a century later, designers first began to think about the load-bearing aspects of buildings in terms of loads (weight), materials and structure. Thinking separately about materials and structures grew during the late-seventeenth and eighteenth centuries, following Galileo's work, and aspects of the internal environment came to be considered independently of the fabric of the building during the late-nineteenth and early twentieth centuries.

It is also worth noting that structural understanding is neither a new phenomenon, nor one that *requires* a knowledge of statics and elasticity. It is a commonly-held misconception that new types of structure and structural forms were devised first by mathematicians or scientists and later taken up by builders, engineers or other designers of structures. In fact, the opposite is the case, with perhaps just one exception —the hyperbolic paraboloid, whose structural properties were discovered in the 1930s. Many children and sculptors display a remarkable understanding of materials and structures without a knowledge of engineering statics, and many structures from long ago show their creators understood the essence of all the basic structural actions better, perhaps, than many engineers today.

A history of structural design would need to address how all the structural elements of buildings were designed. A few examples do exist —for example, beams (Yeomans 1987; Skempton 1956, Sutherland 19), foundations (Peck 1948), retaining walls (Kerisel 1993), and stability design (Mainstone 1988).[5]

Two examples from the history of engineering design will illustrate some of the points made above —the design of Gothic cathedrals and the design of beams for use in buildings.

The design of gothic cathedrals

In some ways, designing a cathedral is not as difficult as it may sound. Most importantly, we already know that it can be done —and this was true in 1200 too, for

many large cathedrals had been built before the period we know as Gothic, especially in France. Nowadays we also know that the masonry is loaded only in compression and that the stresses in the materials are very small compared to the strength of the stone or brick (less than about 10%). The strength of the masonry is, therefore, virtually irrelevant to the success of the structure as a whole.

The structure must, however, be stable. Arches and vaults must not thrust sideways so much that they push over the walls or columns; they must also be curved such that lines of thrust remain within the thickness of the masonry. Also, the loaded elements must not be so slender that they fail by buckling. The stability of a masonry structure depends, then, on the relative size and disposition of the individual pieces. It does not, generally, depend on the absolute size of the building. Hence it is easily possible to build a model masonry structure and scale it up to whatever size you want —the equilibrium conditions for the model and the full-size structure are the same. Nowadays we can demonstrate this using statics; engineers also knew this in 1200, based on their own direct experience and the evidence of thousands of successful masonry buildings.

The exception to these generalities about scaling up from models is the question of wind loads. A masonry structure must not be overturned by winds from any direction. We now know that the force exerted by the wind on an object does not vary in simple proportion to the wind speed. We also know that the speed of real winds increases as you rise further above the ground. It is quite likely that these non-linear effects were the cause of some collapses of early large masonry structures, especially during construction. The main causes of damage to cathedrals were, and still are, in fact, the settlement or movement of the foundations and the burning of roof trusses after lightning strikes.

Something dramatic happened to cathedral and church design in the 12th century. After several hundred years of gradual development since the end of the Roman empire, the designs for large churches took a sudden and sharp change of direction. Round arches were replaced by pointed arches, long barrel vaults were replaced by several discrete structural bays formed by quadripartite vaults (intersecting pointed vaults), the groin vault was replaced by the ribbed vault, columns were reduced in thickness, the maximum area of window in a wall increased from perhaps 30% to around 80%, the horizontal thrusts of the vaults (and wind loads on the roof) were carried out over covered aisles by means of highly efficient flying buttresses. The world had seen no similar transition in building engineering since the 2nd century AD and would have to wait for 700 years for a similar period of dramatic structural development in early Victorian England. During the 400 or so years of the Gothic period there was some steady development and improvement in various elements of the cathedral, especially the slenderness of columns and flying buttresses. However, these changes were relatively small compared to the very sudden developments of the mid-12th century.

The beginning of what we know as the Gothic period can be pinpointed quite precisely as about 1134: in or around that year new building commenced on the cathedrals of St. Denis and Chartres. Within a few years several others were begun, all in the region around Paris called the Ile de France and by the year 1300, some 60 Gothic cathedrals in France and 40 in England, were complete or under construction. The style continued to spread throughout Germany, Italy, Spain and central Europe until the late 16th century.

By and large, we know how the cathedrals were built and we also know quite a lot about how they were designed. We know the names of many hundreds of the individuals who were involved with their design and construction. Unfortunately we have no explicit design procedures dating from the early Gothic period. This is hardly surprising considering the secrecy surrounding the skills of trades such as the mason —it was forbidden to divulge any information outside the masons' lodge, either to other masons or non-masons. We also have direct evidence of what was designed —the buildings themselves. Many documents relating directly to the designs of cathedrals have survived including several sketchbooks of details and plans of cathedrals, the most famous of which is by Villard de Honnecourt. Many more-substantial working drawings survive and we know that some of these formed part of the building contracts.

From such evidence it is possible to work out how some of the plans and shapes have been constructed. They seem to be based mainly on two geometric techniques or manipulations - the combining of various circular arcs and the «rotating of the square»

—a procedure by which a square with an area of half of another can be created by joining the centres of each side. These were used to generate an enormous range of plans and elevations for cathedrals and their components, and several notebooks from the late Gothic period include details of such geometric design procedures. We do have some examples of such methods, although from rather later, contained in some design from the 14th and 15th centuries (Shelby 1979; Sanabria 1982; Addis 1990).

At first sight, such geometric procedures would seem to be of little relevance to what we would now call the structural design of the building. Certainly they were not based on statics. But it all depends on how we look at the matter of design. If we are looking at the history of engineering design at a time when it is meaningless to talk of statics, any design procedures must be of interest, especially in the light of the discussion about the nature of design earlier in this paper. It is also worth looking more closely at what was meant by «geometry» in the 12th and 13th centuries (von Simson 1952, 1956).

Most importantly we know that, somehow, building designers were inspired with a new confidence to push back the boundaries of what was possible and to be much more economical in their use of materials. How did people come to believe that it would be possible to build cathedrals taller, wider, longer and more daring than ever before? What happened, or might have happened to give rise to this confidence? Part of the answer seems to be bound up with geometry and, in particular, with Euclid.

The role of geometry in the mediaeval world

Since Classical Greek times (and probably earlier), people had sought explanations for natural phenomena in order to understand the world they lived in —rainbows, the musical notes made by vibrating strings, the orbits of the stars and planets, the trajectory of missiles, and so on. In every field of what we now call natural science— optics, acoustics, astronomy, music, mechanics, botany —philosophers had explained phenomena in terms of *geometry, harmonics* and *number* which were seen as earthly manifestations of the principles the god(s) had used to create the universe. These explanations were generally expressed in terms of the absolute truths of number and geometry. With numbers there were ratios, squares, multiples, series and so on; in geometry there were circles, triangles, squares, spheres, cubes and so on, and the properties associated with these shapes

This use of geometry, harmonics and number was also incorporated into Christian philosophy. Saint Augustine (354–430) and Boethius (480–525) (both were philosophers and hermeneuts) provided the means. Their works on the science of music, mathematics and architecture sought to demonstrate, using harmony and geometry, the underlying principles of the world as created by God. Further manifestations of these principles were, of course, to be found in the Bible. Augustine took the Biblical passage *Omnia in mensura et numero et pondere disposuisti* (Thou hast ordered all things in measure, number and weight) and applied Pythagorean and neo-Platonic methods to the interpretation of the Christian universe, its creation and its order. Concerning the design of buildings, the Bible also provided dimensional details of a number of significant structures —the Ark, Moses' Tabernacle, Solomon's Temple and the Celestial Temple revealed to Ezekiel in a vision. A well-known 14th century masonic poem even claims that Solomon actually «taught» architecture in a manner «but little different from that used today» and that this science was directly transmitted to France. The writings of Augustine and Boethius dominated the middle ages, and the cosmic applicability of the laws of harmony figures boldly in writings about both music and building throughout the Gothic period.

It was into this philosophical tradition, in northern mediaeval France, that the books on geometry by the Greek mathematician Euclid were suddenly introduced. All copies in Greek or Latin had been lost or destroyed and the books had survived only in Arabic translation. While the English scholar Adelard of Bath was a student of Thierry at Chartres, he visited Domenicus Gundissalinus, the Archdeacon of Segovia, which had recently been recaptured from the Moors. There he came across a copy of Euclid's *Elements of Geometry* in Arabic which he translated into Latin and brought back to his community of fellow scholars at Chartres in the mid 1120s (some say a few years later). The appearance of Euclid seems to have had a profound effect on their work — it has been said that, under this influence from Euclid

«the school of Chartres attempted to transform theology into geometry».

Geometry had, of course, survived as a practical art throughout the Middle Ages and the appearance of Euclid improved the level of geometrical knowledge which could be learnt. Improved geometry facilitated the more accurate *description* of proposed building designs and was of great practical use in the construction process —setting out buildings and enabling the accuracy of the finished parts and their relative disposition to be checked to greater accuracy. Such an improvement alone would have enabled builders to contemplate larger and taller buildings.

However, it was in the capacity to provide *justification* of designs that geometry probably had a more profound effect. Euclid introduced a crucial new ingredient —the notion of the geometrical proof. This provided the perfect tool for the philosophers at Chartres to argue their views logically and to justify decisions made in a wide variety of contexts including, probably, the design of buildings. Just as occurred after the invention of calculus some 600 years later, philosophers put the new theoretical tool to use in every conceivable way and created, quite literally, a new type of geometry —*geometria theorica*.

The distinction between *geometria theorica et practica* was probably first made by Hugh of Saint Victor sometime between 1125 and 1141. In doing so he was looking back to the philosophies of Plato and Aristotle in distinguishing the practical skills from the theoretical (contemplative) skills which had been made possible by the appearance of Euclid. Hugh put the theoretical tool to good use in helping to explain and justify information given in the Scriptures. For instance, he calculated that the reported size of the Ark (40.000 inches) would indeed have been large enough to accommodate all the animals of the world and their food (see Victor 1979, 3 and 32).

There soon followed a number of geometry text books, some purely practical, and others, such as one written around 1140 by Gundissalinus which also dealt with theoretical matters. In this work he distinguished the two geometries according to their respective *purposes* and *duties* (Table 1).

	Geometria theorica	*Geometria practica*
Finis (purpose)	to teach something	to do something
Officium (duty)	to give reasons and dispel doubt	to give measurements or limits which the work should not surpass

Table 1
The several functions of mediaeval geometry, according to Gundissalinus cited in (Victor 1979: 9)

Other 12th century geometry treatises appeared. The prologue to one of these:

> almost makes the practical side of geometry seem subservient or secondary to the theoretical . . . The use of theoretical methods in practical geometry seems to have increased between the twelfth and the fourteenth centuries. At first their role was ancillary to the purposes of practical geometry. Once proofs had found a place in practical geometry, their role increased and changed. Theoretical proof became the goal even of practical geometry. (Victor 1979)

We also find, in the work of the philosopher Robert Grosseteste (c. 1175–1253), just a few decades later, perhaps the first mention of what is now called the hypothetico-deductive method in science, and the principle of falsification whereby a hypothesis should be rejected if it leads to conclusions found to be at variance with experience. Grosseteste typified the contemporary attitude to geometry in declaring that:

> *without geometry it is impossible to understand nature, since all forms of natural bodies are in essence geometrical and can be reduced to lines, angles and regular figures. (Victor 1979)*

In brief, we find the role of geometry in the mediaeval period was rather like the role that modern science —physics, mechanics, materials science, etc.— plays in the world today. It provided explanations of why the world and the heavens were as they were, and how they worked.[6]

The gothic design revolution

To a mediaeval cathedral designer, then, a geometrical model of the building could serve not only as a mathematical model to describe the proposed construction, but also, being based on geometry, it would serve in some manner to justify its adequacy, in conjunction with other engineering knowledge, of course. While we do not know exactly how they used geometry to «give reasons and dispel doubts», the explosion of interest in geometry during the 1120s and 1130s did occur at the very beginning of what we now know as the Gothic era. The cathedrals of St Denis, Sens and Chartres were all designed in the 1130s; another dozen followed during the next 25 years.

Any plausible account of building design in the Gothic era must take account of the significance of geometry in the mind of mediaeval man. Even the scant evidence presented here would indicate that geometry would indeed have helped provide the early cathedral designers with the confidence they needed to propose bolder and bolder designs for the cathedrals.

The changes that occurred in the design of cathedrals during the 12th and 13th century were not technological developments in the conventional sense —no new materials or structural devices were invented. The changes were largely human changes— they went on in the minds of the designers and builders. For this reason it is appropriate to refer to this sort of change as a revolution —a *design* revolution.[7]

These changes were largely complete by the middle of the 13th century and comprised the first of two design revolutions during the Gothic era. The other began a bout a century later when the first signs began to appear of taking into account the weights of materials and the loads these imposed on parts of buildings (Sanabria 1982; Addis 1990).

The design of beams

The second story in the history of structural engineering took place over many centuries —the development of design methods for beams used in structural frames. One key moment in this story was the development of the I-beam and its introduction into buildings in the 1830s.

Beams have been used in buildings for thousands of years for floors and roofs. In classical Greece the choice was generally between timber and stone. Both of these materials occur in nature in larger sizes than they are needed in buildings so extra work is needed to reduce the size down to what would be the minimum possible. It was therefore normal to make a compromise and use a form for beams that was easy to manufacture and large enough to carry the loads. Most beams were, therefore, of rectangular cross section and constant shape along their length. It was well-known that the strength of a beam increased in direct proportion to its breadth and more than in direct proportion with its depth (with the square of the depth, in fact).

To conceive most buildings, therefore, the choice of size was very limited and easy to learn and pass on to young builders. It would depend on whether:

— the beam was in the floor or roof (i. e. on the load)
— the beam was of wood or stone
— the type of wood (or stone)
— the span between the supports

The types of material and the dimensions needed were well known and would have hardly needed to be written down. Nevertheless, as our modern scientific approach to the world developed during the 18th century manuals did start to appear, especially for timber. These gave tables and simple formulae for establishing the scantlings (dimensions) of the timber components of floor and roof structures (Yeomans 1987). There was little or no attempt to justify these dimensions, for they were so well-established.

In fact, there are some very early examples of stone beams from the 5th —3rd century BC in which the cross section of stone has been reduced to less than a rectangle, in order to improve the structural efficiency (Coulton 1977). For the very largest spans— the one in Figure 3, from 400 BC, was 6,2 metres in length - the depth of the beam could be reduced away from the centre of the beam without reducing the strength too much. In modern language, more stone is used where the bending moment is larger and to improve the ration of cross-sectional area to second moment of area. In this case, however, the increased efficiency of the beam was at considerable expense in terms of labour and would only be worth considering for unusually large spans.

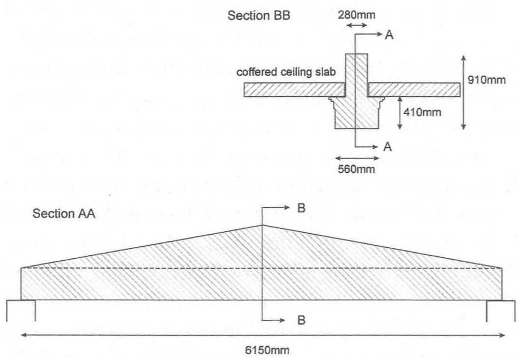

Figure 1
Stone beam

As builder's mathematical skills improved towards the end of the 18th century, some books contained simple formulae which effectively summarised the data previously presented in tables. These were empirical formulae and were not engineering science as we know it —they dealt with only one material and rectangular cross-sections, constant along the length of the beam. The enabled someone to calculate the *relative strength* of a beam-relative, that is to a similar beam but of smaller or larger dimensions. The formulae contained empirical constants with no scientific significance at all. The relative strength was distinguished from the *absolute strength* which was the concern of scientists seeking to *explain* how strong a beam would be.

This situation changed with the introduction of cast iron in buildings, as a fire-proof material for columns and floor beams at the end of the 18th century (Skempton 1956; Sutherland 1984).[8] Unlike with timber and stone, when you make a beam from iron, it was important to use as little material as possible — the more material you use the more it will cost and you had to keep the structure as light as possible, since iron is three times more dense than stone and seven times denser than timber. There was, therefore, for the first time in history, benefit in using the minimum amount of material possible —the search for «minimum-weight» structures had begun and the established deign procedures were unable to help designers in their task— a period of crisis had arrived. This started the urgent search for the most efficient cross-section of beam and shape, from end to end. In fact, the search did not take very long as much of the theoretical and practical work had already been completed by many scientists and mathematicians earlier in the 18th century.

The early application of simple bending theory did not, however, fully reflect the asymmetric properties of cast iron which is much weaker in tension than in compression. By the 1820s another moment of crisis in design procedures had arisen. This led William Fairbairn, with Eaton Hodgkinson, an engineering scientist at Manchester University, to search for new design procedures that would generate the optimal shape for a cast iron beam for use in the many high-rise factory buildings that Fairbairn's firm was building. In this way he hoped to out do the competitors! And he did. The result of the work was the I-beam (Hodgkinson 1831). However, this was not the symmetrical I-beam we now know, since cast iron is six times stronger in compression than in tension. The lower (tension) flange of the beam must therefore have an area six times that of the upper (compression) flange. The resulting section was first used in 1834 in Orrells Mill in Manchester and the earliest surviving example is at the famous Saltaire Mill by William Fairbairn (Fitzgerald 1988).[9]

This paper has reached only the middle of the 19th century in this extract from the story of designing beams, but it continues through the next 150 years right up to the present.[10] Now the mathematical models of beams used by engineers to justify their design decisions are much more sophisticated and can even represent the behaviour of beams under *fire loads* as well as gravity loads, but that is another story.

Conclusion

This paper has sought to demonstrate that the historical development of engineering design is a subject both different and separate from other themes in the historical study in construction.

At present the written history of engineering design is unevenly covered, even though the source material may exist. As yet, for instance, no-one has traced the history of how timber and, later, wrought-iron roof trusses were designed (as opposed to constructed). Rather more serious is the absence of due attention from historians to probably the two most important

subjects in the history of engineering design-the development and use by designers of graphical statics and the factor of safety. These are themes in need of research.

Unfortunately practising engineers often have little time or inclination to follow these directions, so it must be hoped that some professional researchers have been stimulated by this paper to take up the challenge.

Notes

1. See, for instance, Bowley 1966 and Nisbet 1997.
2. For further discussion of these issues, see Addis 1990, 1994, 1997 and 2001.
3. The profession of building engineer is hardly known by the public. When building is mentioned, people usually think of the architect; when a building engineer designs something, the press refer to him as an architect. This has been my experience during my whole life. I am of the opinion that the building engineer himself is largely to blame for this state of affairs.
4. L'art de l'ingenieur-constructeur, entrepreneur, inventeur at the Centre Pompidou, Paris, 1998. (Picon 1997).
5. These and other examples are reprinted in Addis 1999.
6. These issues are discussed in depth in Addis 1990. This book is now out of print, but a few copies are available from the author.
7. This phrase is drawn, by analogy, from the idea of the scientific revolution that the American philosopher Thomas Kuhn developed some thirty years ago and which has transformed our understanding of the development and history of science (Kuhn 1970, Addis 1990).
8. These two papers are included in Addis 1999.
9. This paper is included in Sutherland 1997, along with Sutherland 1990.
10. A further episode in this story, the design of steel beams using plastic theory, is given in the author's other paper in this conference: *The nature of Progress in structural engineering*.

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The nature of progress in structural engineering

Bill Addis

Progress is nowadays an emotive word intimately linked with the idea of becoming better. It is embedded in our way of thinking about technology whether in construction, aviation, medicine or computing.[1] Its central role in our economy has inevitably led to a search for the mechanism by which progress occurs: if the mechanism is understood, it can be influenced in both speed and direction. Nevertheless, to date, this goal has not been achieved. Some would have it that progress in practical technology is a consequence of theoretical study (Hall 1969); while others would assert the opposite (Baker 1936–37; Addis 1983). Another theory would have it that progress is often by a process of diffusion from areas (or industries) of high technology to areas of low technology, rather like electric charge flowing from high to low potential.[2]

Yet models of progress such as these do not seem to describe the experiences of the practising design engineer in the construction industry. A pragmatic approach is taken by some who portray progress mainly as learning from mistakes (Blockley & Henderson, 1980). Others have focused on the decision process during design development that is essential to ensure the design work on a particular project progresses in a converging manner (roughly) linear direction, and not in a cyclical way (Happold et al. 1976). Design is usually about choices and progress needs to be seen in the light of the outcome of such choices —will they bring about an improvement of any kind? and, if so, for whom is it an improvement?

Much progress has its roots firmly in the profit motive —creating a product or service that people want to buy, and ensuring it is better or cheaper than those of the competition. Economics is seen as the sole driver for the process of progress.

In the historical study of engineering and technology it is important to have other, non-commercial criteria of excellence in order, for example to assess whether a certain culture is more technically or technologically advanced than another. In construction engineering, the indicators of such performance might include the following:

- materials technology: the materials being used, their quality, methods of manufacture, shaping and joining;
- the accuracy and consistency with which structures and buildings were made; measurement and surveying skills;
- types of plant and machinery used during construction;
- the skills of the various those working on the design and construction of a project;
- indicators of technical achievement: length of span, height of a structure, economy, slenderness or weight of a structure; structural ratios (span-to-rise ratio of an arch; span-to-depth ratio of a beam; specific strength or stiffness)
- methods of design: their sophistication and precision, their generality, their reliability;

— scientific knowledge: the nature of the knowledge and its role in the production of buildings and structures.

Underlying most of these issues are various thought processes and ideas. They have an intellectual side to them, as well as physical. Yet this aspect of construction engineering has not been given the same attention as in other fields of human endeavour —the history of ideas, in general, covers all of mankind's intellectual activities— mathematics, science, medicine, music, politics, painting and sculpture, philosophy. In all of these fields of activity, it is considered vital to be able to evaluate the inherent worth of ideas and to compare one idea with another. This approach is not often practised in engineering.

Yet, both individually and when taken together, the indicators given above can help us develop ideas of excellence in construction. This, in turn, can enable us to make an assessment of the historical worth of an individual building structure and to compare the relative merits of two different structures or buildings. This will help us assess their place in the progress of the art of construction. Likewise, such an approach can help us assess and compare the skills of individual engineers and builders, and the value of their contribution to developments in construction history.

The philosopher Thomas Kuhn proposed a way of looking at the development of sciences such as astronomy and chemistry which takes a very human view of the work of scientists (Kuhn 1970). He describes the workings of scientific communities who, at a certain time, share and subscribe to the same set of beliefs about their field. By and large they work on problems they are confident they will be able to solve. From time to time, however, they encounter problems they cannot solve, or phenomena they are unable to explain («anomaly» in Kuhn's language). Generally they find they can overcome these problems by making minor revisions of their theories and beliefs. Occasionally, however, they encounter a problem that cannot be explained with current theory —indeed it seems to challenge received opinion to its very core. The classic example Kuhn uses is the Kepler's theory that the planets orbit the sun rather than the classical view that the earth was the centre of the universe. This leads to what Kuhn calls «crisis» and, finally, the supplanting of the former theories by a new, incompatible set. Because the process is highly disruptive, representing a discontinuity in developments and is largely intellectual in nature, he calls the change one of revolution —a *scientific* revolution.[3]

Kuhn's ideas have found application far beyond the history of ideas in science and can also help to bring an understanding to progress in the intellectual aspects of engineering, in particular, in design methods. In place of the scientists' scientific hypothesis or theory, the parameter which encapsulates the design engineer's intellectual tool is the Design Procedure. This is the process by which the engineer selects the appropriate engineering knowledge —both practical and scientific— based on his experience, and uses it to generate the key outcomes of the engineer's work:

— a *description* of what has to be manufactured and built: materials, specifications, dimensions, construction method. This may be a verbal instruction, but, especially when the building is large or complex, would usually be written down or in the form of a drawing or a model; and

— a *justification* or *explanation* of the design proposals that have been made. This may be achieved on the strength of precedent, full-scale tests, experiments on small physical models, various calculations, or the use of mathematical models, nowadays based on engineering science.

The Design Procedure may be represented as follows:

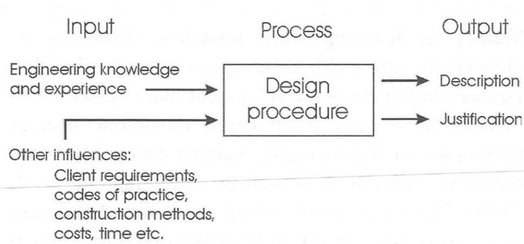

Figure 1
Design procedure

By looking at the world of engineering design in a way similar to that used by Kuhn to study the activities of scientists, we can propose, by analogy, a model of progress in engineering design where design procedures takes the place of the scientific theories. Generally, any anomalies encountered in «normal design» can usually be explained and resolved, and minor adjustments made to design procedures. Occasionally, however, serious anomalies are found which lead to a state of «crisis» and, finally, a design revolution.

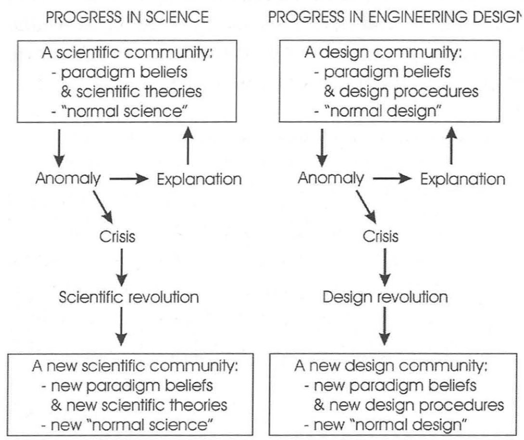

Figure 2
Design Revolutions

Two examples of design revolutions are given below which illustrate how progress occurs in engineering design methods —the design of steel frame buildings in the 1930s and the design of tension structures in the 1950s and 1960s.

THE DESIGN OF THE STEEL FRAME

During the development of design methods for columns and beams in the structural building frame, three different design approaches were developed during the 18th and 19th centuries:

— ensuring that a structural element could carry a certain load without collapsing
— ensuring that a structural element could carry a certain load without deflecting or deforming more than a certain amount
— ensuring that the material used was never stressed beyond a certain value

Developments in structural engineering design methods have been largely the story of these three approaches and their perceived benefits. With hindsight it is easy to observe that all these approaches have been used with success, but it did not always appear so clear at different times in the past. The crucial issue is *how* they were used. Each depends on several very different types of engineering knowledge:

— the mathematical models used to represent the loads on a structure
— the statical models and mathematical techniques used to represent and analyse the idealised structures
— the internal distribution of stresses in an ideal material
— how the mathematical models of the materials used were adapted to real materials with their flaws and variability inevitable in all manufacturing techniques
— the various empirical constants, factors of safety, and load factors that are used in all design procedures in order to make the theoretical models useful to the design engineer.

One example from this history will serve to illustrate the often intense disagreements that have occurred between the various people involved —the birth of the so-called «plastic» approach to designing steel structures during the 1930s and early 1950s.

During the early decades of the century steel structures were designed to work at a certain «working stress» at a level well below the yield strength of the material. In addition, the «real» loads were increased by a factor of «safety» to add the engineers' confidence that the resulting structure would be satisfactory in every way.

Gradually, however, it was observed that one of the fundamental assumptions of this design method was frequently, or even always, incorrect —namely that the stress in a new steel beam or column was zero when leaving the steel rolling mill. It was found that

the stresses locked in by the rolling process and the cooling from red heat to room temperature were often 30–40% of the yield stress, and sometimes nearly at the yield stress. This dented the confidence of many engineers, including those working in the Steel Structures Research Committee in England in the 1930s. It seemed that most steel structures were being designed to carry stresses greater than the material could carry —a crisis of confidence, indeed. However the investigations which followed did not solved the problem, they exposed many more causes for concern. The words of one of the principal investigators, John Baker are illuminating:

> Designers of steel frame buildings are recognizing more and more clearly that their methods of calculating the stresses in the members of the frames are largely empirical . . . It is common practice, when considering the effect of floor loads, to assume that all the beams are simply supported on brackets attached to the faces of stanchions. When wind loads have to be borne, however, this assumption is considered to be unsatisfactory, and it is then customary to imagine a perfectly rigid connection between each beam and stanchion . . . Though apparently leading to simple methods of calculation, these patently contradictory assumptions, by failing to give a true picture of the stress distribution in the frame, actually give rise to fresh problems, such as the eccentricity of loading and the `equivalent length' of a stanchion, which have no solutions (Baker 1932, 3).
> [The structural engineer] knew that some account had to be taken of eccentricity of loading, but one of the most disputed points in the method of design was what to assume (Baker 1954, 1).
> It seemed probable . . . that [the method of design] was in the nature of an empirical formula in which the numerical factors, appearing in the regulations under the guise of superimposed loads, working stresses, eccentricities and end-fixity, were merely constants justified by experience (Baker 1954, 2).
> In the present method of design it is assumed, when estimating stresses in members, that all members are loaded. This may not produce the most rigorous stress conditions (Baker 1935–36, 136).

In summary, the SSRC confirmed that the design procedures of the time were *irrational*. This had a most important consequence:

> Simplified assumptions must always be made by the designer, but if those assumptions are so sweeping that the true behaviour of the frame is disguised, then economy of material and evolution of the method of construction are impossible (Baker 1935–36, 179).
> [The] review of existing regulations left a strong impression that the method of design of steel framed buildings in common use had no firm rational basis, and that the wide range of constants representing loads and stresses made it difficult to justify even as empirical. It was clear that no advance could be made until the real behaviour of this form of structure under load was understood. (Baker 1954, 6).

The first response to this awareness of problems with the existing design procedures was, naturally, to persist with the elastic design philosophy as a route to avoiding the contradictions and to breaking down the barriers to progress. The first step was an immediate response to the perceived emergency:

> So unsatisfactory . . . were the rules governing the use of steel in buildings that it was felt desirable to draw up recommendations for a Code of Practice [later BS449], based on . . . the available knowledge and experience of practising engineers (Baker 1935–36,128).

The second step was to discover how framed structures actually did behave; this was the major work of the SSRC. The results were startling:

> The investigations . . . show that the stress calculations made in design today give a very faulty representation of the distribution of stress in a frame. What is more important, however, they disprove the usual assumption that the worst conditions are provided for each member when every member carries its full load (Baker 1935–36, 178–179).

These events in the 1930s are a clear example of the crisis of confidence that is characteristic of a Kuhnian revolution —an established set of beliefs has become untenable. The resolution of the crisis occurred after the Second World War when a new design approach for steel frames was devised which took account of the plastic behaviour of steel and avoided the logical inconsistencies that had been exposed in the earlier study of the elastic design methods (Baker 1954; Baker et al. 1956).

THE DESIGN OF TENSION STRUCTURES

In 1952 Matthew Nowicki designed the first substantial pre-stressed tension structure —the roof

of the Raleigh Livestock Arena in North Carolina. It was doubly-curved and consisted of two, virtually orthogonal sets of steel cables stretched between inclined concrete arches and was clad with steel sheet. The shape of this roof was simple enough, and the steel cables were few and regular enough for the roof to be calculated as an elastic, prestressed structure —a type of design procedure which was familiar to those engineers working with the prestressed concrete structures which were currently enjoying considerable popularity (during the late 1940s and early 1950s).

The Raleigh Arena proved to be a tremendous inspiration to many architects, including Eero Saarinen and Kenzo Tange who immediately wanted to exploit the architectural possibilities of tension structures. However, they found their imagination severely frustrated by the very restricted range of geometrical shapes for which the engineers were able to undertake calculations —the calculations could only be done for surfaces which could be expressed as a mathematical equation. The early large tension structures of the 1950s thus really still belonged to the engineer's world of freely-hanging structures, prestressed concrete and the shapes of reinforced-concrete shell structures such as the hyperbolic paraboloid (Forster 1994).

The architect Frei Otto was impressed by the possibilities of doubly-curved, prestressed surfaces and started experimenting with a variety of models to generate a wide range of curved shapes and surfaces formed by chains and nets in tension. Many of these were direct descendants of the hanging chain model which Robert Hooke had identified in the 1650s and used again by Gaudí for the design of his cathedral in Barcelona.

Otto also experimented with a new type of structure which relied on the tensile forces within a sheet of fabric, taking his inspiration from the shapes of soap bubbles. Using elastic sheets and a variety of nets including ladies' stocking material, he was able to generate an enormous range of natural shapes which had no convenient mathematical equations —they were, literally, inconceivable in mathematical terms and hence could not be analyzed by engineers. It was particularly frustrating to an architect to be able to conceive and generate beautiful shapes, in miniature, which the engineers were not able to design at the sizes required for use in buildings, simply because the surfaces did not have convenient mathematical equations. This state which existed in the late 1950s was effectively one of «crisis», in Kuhn's sense of the word —a line of progress could be imagined but not achieved using the conventional approaches. There was, quite literally, no way in which current structural design procedures could be used or developed in order to enable such structures to be designed.

The crisis was resolved by taking a radically new approach and devising design procedures very different from those used to design the large engineers' cable tension structures, such as the Raleigh Arena. A new type of design procedure would have to be developed which relied entirely on models.[4] Two problems had to be solved. The first was to develop the technique of making models which were believed to be a suitable representation of the sort of materials and structure and which would be used in a full-size building. This was achieved by experimenting with three types of material: a variety of inelastic chains, nets and fabrics (giving models whose geometry was governed only by statical considerations), various elastic sheets, and the purely plastic material of a soap film (the surface stress in a bubble is independent of its size) (Bach 1988).

No one of these model materials was found to be an adequate representation, and often all three types of model had to be made to generate different types of information. The soap film models presented the greatest technical challenge —no-one had ever made bubbles up to a metre across and which would last long enough to be studied and measured (soap solution was continuously fed into the bubble to balance evaporation losses). The second problem was to devise ways of accurately scaling up the geometry of the model, perhaps several hundred times, in order to provide the material and connexion specialists and fabricators with sufficient information for them to be able to achieve the desired results. A variety of techniques was developed to enable the complex geometry to be accurately surveyed. Chains and nets were not too difficult, but fabrics, elastic sheets and bubbles required the drawing or projecting of regular grids onto the surfaces as well as the use of careful photography. Sometimes a solid cast was taken of the shape made by a fabric or elastic sheet to allow small pieces of flat material to be laid on them to help plan the best cutting patterns for the tensile material in the full-size structure.

The justification of the proposed designs was, in one sense, achieved simply by demonstrating that the model structure would work —the very geometry of the tensile shape is governed by the statics and elasticity of the materials of which the model is made. There remained only the not-inconsiderable problem of scaling, by a factor of perhaps several hundred, the anticipated loads and elastic properties of the materials between the model structure and the full-size version, and vice versa. This was achieved and a number of trial structures (spanning only a few metres) were built and tested in order to establish the effectiveness of the design procedures.

Designs progressed rapidly from spans of a few metres to the 40 metre spans of Otto's German Pavilion at Expo'67 in Montreal and the 135 metre spans of the roof over the sports stadia at the 1972 Munich Olympics. This whole class of structures had only become possible by developing an entirely new type of design procedure which avoided the impasse inherent in previous, wholly analytical design procedures. They could, quite literally, not have been designed in the 1950s, although they could, in principle, have been manufactured. The tension structures revolution of the 1960s was not due primarily to developments in engineering materials.

Once the new design procedures and the new types of structures had been created, so the engineering scientists were able to proceed in new directions. Now able to gain a qualitative understanding of the structural behaviour of the new structures, they could proceed to create mathematical models to simulate it —yet another instance of practice leading theory! As it happened, during this same period (the 1960s) the computer was becoming more widely available and it became an essential tool in dealing with the complex mathematics behind the geometry and non-linear structural behaviour of soap bubbles, nets and elastic sheets curved in three dimensions.

Nowadays computer software for tension structures is able to display to the designer the complex geometry of tension structures and also, by means of colour-coded output on the screen, to give visual feedback of how highly stressed different parts of the structure are. Using this highly interactive technology a designer almost has the feeling of building a real model using a soap bubble or elastic sheet. The designer is able to choose different support conditions and surface geometries and be informed immediately of their effect on stress patterns. Finally, the computer is able to help calculate the most suitable cutting pattern which the manufacturer will need to build up the curved surfaces from a series of flat pieces of fabric and cables of precisely calculated lengths. The computer has thus enabled designers to develop further the building and testing of models as part of a design procedure which simultaneously provides both the justification and the description of a proposed design, a process fundamentally the same as that devised by Otto.[5]

Conclusion

This paper has argued that progress in engineering comprises both practical and theoretical or intellectual issues and that the intellectual aspects have not been widely studied. The paper has considered the intellectual nature of the engineering that is incorporated in the engineering design process, and has drawn from other spheres of intellectual activity, notably science. Using a methodology developed within the history of ideas for understanding the nature of progress in science by Thomas Kuhn, the author has demonstrated the improved understanding of progress in engineering that can be derived.

Notes

1. This has not always been the case. The idea of progress was created in the 19th century as a means for encouraging economic development and wealth at a national scale and profit for private forms (see Peters 1981).
2. This process is often called «technology transfer» though is not always found to be a reasonable or respectable process (see Addis 1992 and Harris 1998).
3. This process is discussed in more detail in Addis 1990. This book is now out of print, but a few copies are available from the author.
4. This was not, of course, the first time that scale models had been of help to design engineers —see, for example, Smith 1976–77 and 1977.
5. This design revolution and the design of membrane structures are discussed in more detail in Addis 1990, 1994 and 2001.

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Reconstruction of the oldest cable wire suspension bridge in Italy «Leopoldus II»

Alessandro Adilardi
Salvatore Giacomo Morano
Paolo Spinelli

Close to Florence, in Tuscany, a very old suspension bridge: «Leopoldus II» from the name of the Grand Duke; is probably the very first wire suspension bridge in Italy and one of the first in the world. In Italy earlier only some small pedestrian structures are historically recorded.

The first suspension bridges were designed in England and in France. As the Grand Duke of Tuscany Leopoldo II knew that, willing to be ahead with the progress, he sent Alessandro Manetti, his best engineer, in France. For this reason, the experiences about French suspension bridges were revealed through the Manetti's direct knowledge, recorded in an ancient day diary (see references).

When Manetti was back he received the assignment to design a cable wire suspension bridge in Poggio a Caiano, (Province of Prato, nearby Florence) next to the Royal Palace, to provide a new way out for the Grand Duke to his lands, on the other side of the river Ombrone. The bridge, constructed in 1833, had three wire cables for each part and a wooden deck, and is remembered as a work of art, as results also from photographic documentation of years around 1935. Unfortunately the German Army destroyed it under the World War II while they were retiring.

Now it is possible to see on both sides of the river the monumental masonry piers. In lately years interest is growing for a restoration of the bridge. Different levels of renovation/restoration are presented in the paper:

1. conservative maintenance. This is possible where we still have the structure and higher loading condition is not required
2. reconstruction of some parts of the bridge with new materials

The paper starts with a documentation of the design of the old bridge based on historic documents, presents a verification with numeric models of the old structure and the new one, and at last proposes different design strategies for the reconstruction of the bridge.

Introduction

At the beginning of the 19th century, thanks to the progress and to the industrial revolution, steel started to be used in civil construction. The advantages of the steel, even from an aesthetic point of view can be seen in suspension bridges. In this kind of structures it is possible to obtain lightness and elegance that were before impossible. In fig. 1 it is possible to see the bridge, which was the first wire suspension bridge in Italy. The bridge represented the advanced technology, the will to introduce scientific innovations in the art of construction. The Grand

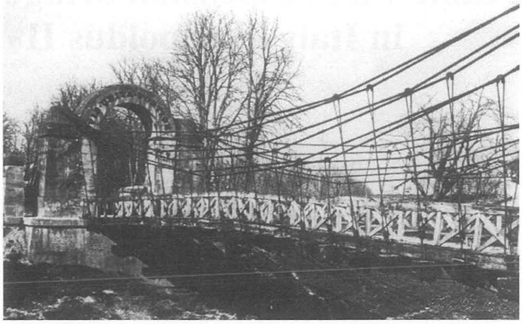

Figure 1
The bridge «Leopoldo II» 1931 —Belli Archive

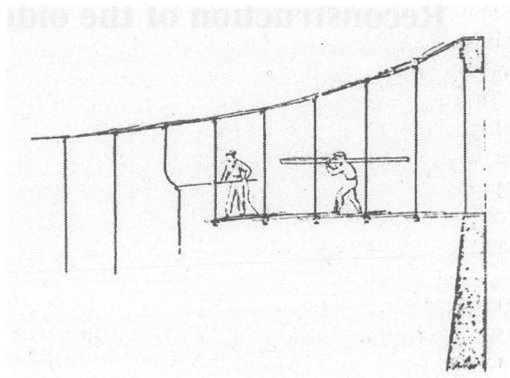

Figure 2
Construction of a suspension bridge

Duke Leopoldo II wanted the construction of the bridge in order to be not inferior to the other Sovereign in Europe. The «Leopoldo II» Bridge was constructed in 1833 and was used until the World War II. Now only the powerful masonry arcs remind us of his presence and, even though after the World War II they were forgotten, they now capture again our attention although of its small dimension. The bridge has always provoked great admiration and astonishment in the people who crossed it.

The paper presents the story of the bridge and of the people who wanted it. Through the knowledge derived by a deep research in historical archives, we tried to understand the reasons and the will that regarded it. The experience and the knowledge that were available at that time are analyzed.

A proposal of reconstruction is made paying attention to the philosophy that was in the previous construction.

Beginning of engineering science

At the beginning of 19th century there was not a definite typology of Engineer. Men who were both architect and engineer designed important constructions. Most of the time, the dimensions of the structural elements were determined by experience rather than by calculation. The industrial revolution had already started in England; technology was therefore growing up. From the point of view of bridge-construction history, many innovative techniques and materials were coming out. The new material, steel, gave the possibility to build structures that were impossible before, thanks to its tensile capabilities. The mentality of that age, orientated to the future, gave confidence to the new type of structures.

There were many problems for the designers of new typology of structures. Information could only travel through books and very often we could find architect-engineers travelling in Europe or outside it to find new answers. We have seen Navier, sent by Louis XVIII, in England to learn about new bridges that were built there. From Italy the Cavalier Luigi Giura was in the same lands where Navier had been before to design his suspension bridge on the Garigliano in the Reign on Naples. Ellet visited France in 1831 to see the new suspension bridges made with wire cables instead of iron eyebar chains, then he returned to America to realize the Weeling Bridge, the so-called «Thousand-footer», for its dimension. During the same years we can also see Alessandro Manetti, who will design the suspension bridge in Poggio a Caiano, travelling around France under the order of the Grand Duke Leopoldo II. During his travel he took all the new technology used in that country not only for suspension bridges but also for other construction techniques. When he returned to Italy he presented a report of what he had seen to the Grand Duke Leopoldo II. Travelling was very uncommon during those years, people were not

used to do it, and important architect-engineers were doing something special for the developing of the engineering science.

The suspension bridge can be seen as a product of the industrial revolution. After the first pioneer realizations, this kind of structures had an incredible growth. The key of the success of this kind of bridges has to be found probably in its simplicity and the possibility to cover distances that were impossible to cover before that time. Before that time the structures were designed only using empiric proportions. With the coming of suspension bridges simple calculations started to be made, simple formulas based on the concept of catenary can be applied. Wind and second-order effects were not taken into account; hence the decks were generally without stiffness. Before the beginning of the 19th century, the art of construction was given only by means of experience. With suspension bridges mathematical modelling started to be scientifically applied to structures: engineering was being born.

In Europe there was a great diffusion of modern suspension bridges, which was partially due to Claude Navier; after his journey around England he wrote a manual about this kind of structures for the «Ecole Nationale Des Ponts Et Chaussées» of Paris. He revealed some formulas to calculate a bridge without stiffening deck. Considering for the cable a shape of parabola he has found the today well-known formula

$$H = (w \cdot l^2/8 \cdot f)$$

From which we can calculate the horizontal component H of the tension in the cable. For the meaning of the other elements l is the main span, w the weight for unit length and f the sag. The theory of catenary was already solved by Bernoulli in 1691, and the formulas of the parabolic cable were found out by Nicolas Fuss in 1794. Navier extended these theories to include the effect of variable loads. He has studied the variation of tension in the cable loaded with a concentrated force P on the midspan, founding

$$h = 3 \cdot Pl/f \cdot 1/16$$

and the maximum deflection

$$v = P \cdot f/2 \cdot wl.$$

Both formulas can be found using the hypothesis of having an inextensible cable. The greatest problem of a structure designed in this way is the lacking of global rigidity. Some designers tried to increase the rigidity of the cable to control the deflection of the deck. In some other solutions the cables were substituted with rigid frames.

Through the analysis of his formula, Navier suggested to have a ratio between sag and span of 1/12 and not more. A bigger sag increases the maximum deflection of the bridge. The formula also suggests that another way to limit these phenomena is to increase the self-weight of the bridge; this was a surprising result for that time. Just looking to early structures and their behavior was possible to understand the help of a stiffened deck. Even if not fully understood often was used a stiffening truss. In America Finley, who was the first to have done a patent for suspension bridges, suggests in his patent to build this kind of structures with a stiffening truss. His suggestions where received by the European engineers. On the contrary, Ellet suggested to build bridges with a completely unstiffened truss because this is the *«simplest form»* we can use, and to have stiffened deck bringing an *«artificial rigidity»*.

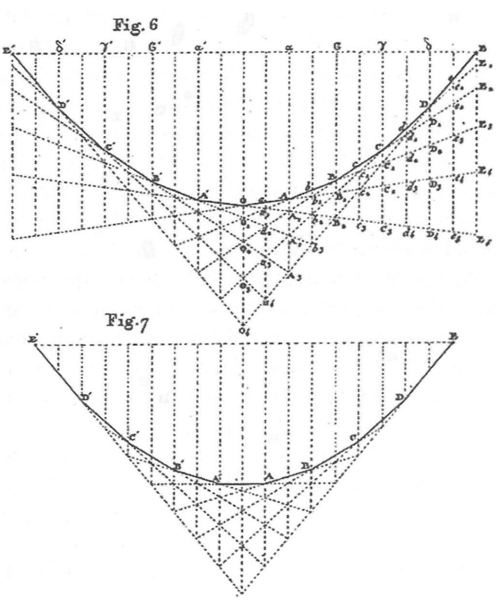

Figure 3
Study of parabola made by Navier

Big attention was used on the realization of cables; the most common wire used in Europe, was generally the number 18 with a diameter of around 3 mm. The wire was made by the use of the Draw Plate, a plate made with progressively smaller holes with hot cast iron pulled through the plate. The mechanic characteristic of the material was controlled by traction tests and the resulting thin iron wires had a good resistance even if the material was not very homogeneous. The most common problem was the loss of resistance caused from folded wires.

Various systems were used to connect wires together to form the cable and to connect the cable to the anchorages. One system used to connect the suspenders with the main cables was the use of saddles, trying to keep the curvature of the cable as wide as possible.

One of the most interesting aspects of the construction was the disposition of the cables. It was not uncommon to have each one disposed with different sag and in the meantime inclined in respect to the vertical plane. The reasons of this disposition can be found in the search of a better rigidity of the structure and, not less important, a better view of the structure. See all cables disposed in different levels and inclined gave a nice sensation to the one who was entering.

People at the beginning of 19th century were not completely ready to see such light structures, and various systems were used to try to give the impression of a bigger stability. Potent masonry arc where built on the entrance of the bridges. On the Menai Bridge, in England, impressive arched piers support the two lateral sides.

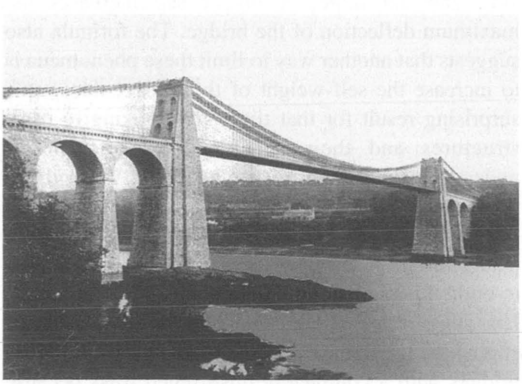

Figure 4
Menai Bridge

But the beginning was not impervious to errors; the early structures collapsed few months after the construction, because of the wind or just few people jumping over the bridge: enough to cause the collapse. In a period of big innovation a collapse is a deterrent to the diffusion of the technique, but also an incentive to perfection. The most bigger problem encountered at the beginning was the oscillation of the structure and various way to limit this phenomena were tried. We have seen cables placed under the deck, stiffening parapets or just the use of supplementary stays. In 1830 Capitan Samuel Brown built a railway bridge over the river Tess, but after the construction the train caused big oscillations on the structure, Robert Stephenson wrote «Wave before the engine [. . .], just like a carpet». No mathematic modelling existed at that time to explain vibration of the suspension bridges. Thus, only in the second half of 20th century we were able to partially explain the motions of those structures.

A treatise on static was born only in 1859 with the Rankine theory where the cables were considered inextensible. This was because of the simplified mathematic involved with this assumption. But it was only in 1888 that Melan laid the formulation of deflection theory. The theory, called also the perfect theory, take into account the change in shape of the cable under live load. The longest suspension bridges in the world were built according to this theory; it was only with the advent of computer that changes were made in the design process, most of them calculated without the use of computer, even though the iteration included are time consuming.

At the beginning of the 19th century the voices about the new structure were starting to travel around. All the Sovereigns in Europe wanted to be updated about the new technologic progress. There was a widespread trust on new technologies.

History of «Leopoldus II»

In 1831, Leopoldo II di Lorena not wanting to be inferior to the other sovereigns in Europe, sent his best man in France to study what they were doing there. Alessandro Manetti after his journey exposed what he had seen there and the decision of the construction of

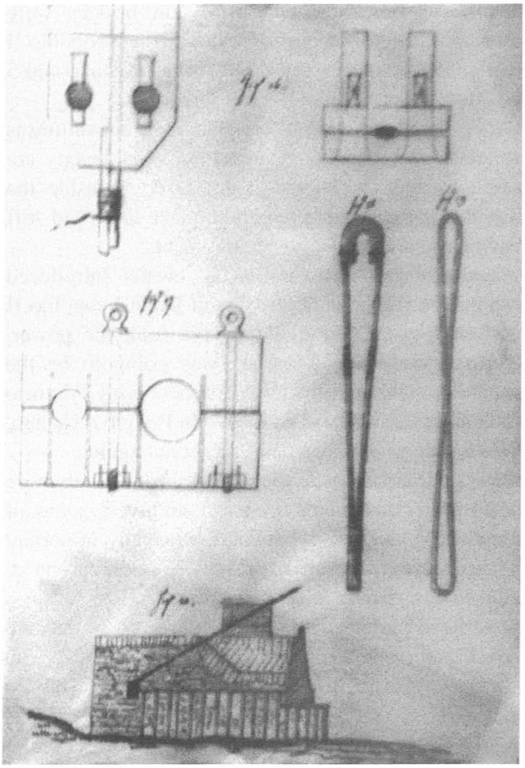

Figure 5
Relation of Manetti to the Grand Duke Leopoldo II of Lorena

the first wire suspension bridge in Italy came out. On autumn 1833 the bridge «Leopoldus II» designed by Manetti was built up. Manetti writes: «I made them built a suspension bridge with hemps of iron wire on the Ombrone in the park of Poggio a Cajano [. . .], it was the first bridge of that kind that we made, suitable for the step of the coaches, and those which it was destined belonged to the Monarchs and the Court». The magnificent opera were used by the Lorena to move from their lands to the Villa del Poggio, the house they used to live. On the other side of the river there was the park of Pavoniere, the English garden with canals and ludic places where the Sovereigns used to bring their guests and come back from the suspension bridge to show what they where able to do.

Basic simple rules have been applied to the construction of the bridge: the masonry foundation of the anchorages of the wires must be connected to the foundation of the piers, the wires must be far from each other in order not to be differently influenced by the temperature and carry equally the weight of the bridge, and the piers that sustain the cables should be connected by arcs not only for a matter of solidity but to give the impression of power (*«Hanno la figura di una potenza»*).

Many years after the construction of the bridge, Alessandro Manetti, in his memories published posthumous, said that a bridge like that shouldn't have been built because of the small span. Suspension bridges have to be built with bigger span to exalt the power of these structures. The «Leopoldus II» seems to be too much powerful for the kind of structure it is.

The wires where made at Follonica were there was the royal industry of cast iron. About 50 thin wires of a metal in between steel and cast iron formed the cables. In total 12 cables sustained the bridge, 6

Figure 6
The Bridge Leopoldus II in 1980, Archive Gradi

cables for each part, inclined respect to the vertical plane to give transversal stiffness to the structure.

The wires of about 3 mm of diameter are placed one by one over the bridge to form the cable. Each wire was pretensioned to eliminate defect and to test its tensile strength. Every 30 cm of the cable were rolled up a thin wire under hot temperature.

A long search in the national archives was carried out to find information about the bridge. On the National Archive of Florence some documents where found with some indication about the renovations made to the bridge during the first years. The truss was made by oak, a wooden structure like this requires to be kept during years. Some voices about renovations of the truss or the parapet have been found in the books of the economic entrance of the royal land. After 1841 the bookkeeping book is clearer and it is possible to infer a big renovation every 10 years. The treatment of the cables is interesting: they were very often painted, about every two years, with a mixture made of oil and vegetable substances. This special treatment was to prevent the cables to be attacked by the water or other atmospheric agents. Sometimes the cables were painted in white. Very often the stone pavement lying just before the entrance was renewed. The wooden truss is a more flexible material than the stone of the pavement; vehicles while entering the bridge made a slow opera of erosion.

On the 11th September 1849, 25 lire are paid for a stuck of steel that have been stolen. This can make us reflect about the value of the steel at that time. The metal was precious and suspension bridges were often controlled 24 hour to 24 hour. The Leopoldo II Bridge was inside the land of the Grand Duke and 5 guardians controlled the whole territory.

The greatest part of material used for renovations is generally wood. The principal wooden beams are made by oak, the beams were inserted inside the water of the river for 10 years to make the wood stiff and resistant to the atmospheric agents.

In 1859 many revolutionary events introduced changes to the power. The Grand Duke Leopoldo II was sent to exile and the Savoia took the power. During those years Florence was going to be the capital of Kingdom of Italy. The new King, Vittorio Emanuele II, soon started to live in Poggio a Caiano, whose lands were cured and preserved again.

All documents about the renovations made in those years were discovered in royal archives, some of them still hidden or not well know; many important papers were found in the Archive «Guardaroba di Palazzo Pitti». In the discovered papers it was possible to find detailed renovations of the bridge and the dimensions of the various elements of the structure. Through this analysis it is possible to propose a reconstruction able to respect the original proportion of the all elements. When the old structure is not anymore present, a valid reconstruction should always been supported by an historical analysis. Through the documents it is possible to know and to understand the proportions of the structures and the reasons behind them.

From the papers of the Savoia archive was also possible to see the beginning of bureaucracy of

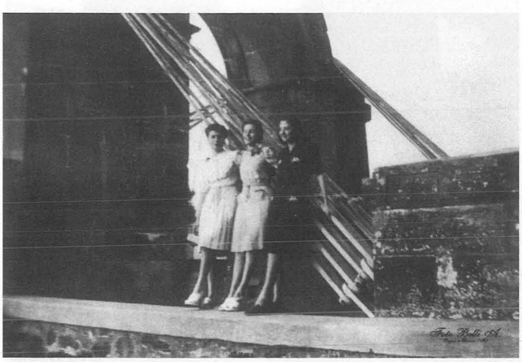

Figure 7
An amateur photography, probably taken in 1838. Archiv Belli, Poggio a Caiano

Figure 8
The bridge today, 2002

constructions. Generally a big renovation is made by the company who has won an auction made with a special game called of the «three cards». No one knows how that system worked, but we can see that during those years it was always the same company to win. The beginning of the work is stipulated by a «report of beginning» in the papers with the date of beginning and the description about the duration of the work. On the end is stipulated the «Report of finishing», which name is clear itself. Interesting is the «Act of submission», where the responsible of the work were taking the engagement of finishing the work in within the money of the estimate. We should say that the estimates generally were always much more than the money really needed at the end of the work, not properly the same of today!

By the way the bridge Leopoldus II was renovated very often. The Villa of Poggio a Caiano and all the lands annexed remained property of the Grand Duke until 1919. After that time the property was donated to the State and this was the beginning of the end. Just after 2 years in 1921, the secular unity of the lands was broken and part of the territory, included the suspension bridge, was donated to the National Veterans Organization. After some years of bad administration big part of property was sold. When the damage was done there were some attempts to save the bridge by the State Superintendents but nothing went out. It seems that a renovation of the bridge was done in 1938, but information are scarce and contradictory. Some witnesses from the old people of the area around said that the bridge was in bad condition before the War. The World War II finished the job: during the retire of Germans, in September 1944, the bridge was mined at anchorages and fell down into the water.

Time goes by and the last decades are enough to let the structure be forgotten. No trace is there anymore of the cables and the truss. Only the monumental masonry portals are there to remind us of the existence of the bridge.

Reconstruction

Reconstruction, more than a renovation, puts forward a number of questions to be solved. Does the structure have to be rebuilt exactly the same way as it was? Or is it only the philosophy that was behind the structure

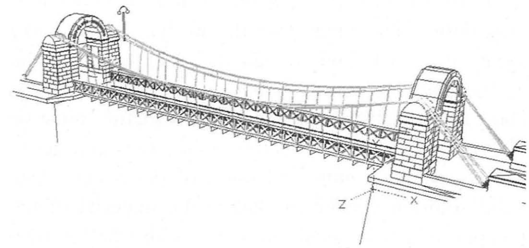

Figure 9
Proposal of reconstruction

that has to be maintained? Very often the loads for which the structures were designed were generally much lighter than the ones that the bridges have to carry today, and even the hydraulic imposition can be different. These conditions obligate to change the stiffness of the original structure. Changing the original structure can be made maintaining the philosophy of it. Using different materials can be one solution. Thanks to modern technology, resistance of modern outfit can be much higher than the one used for the original structures.

A courageous way of thinking can be the separation of tasks. It is possible to create a main structure with modern and technologic materials and to maintain secondary elements as they were before.

For the Leopoldo II Bridge evaluations are made with collected data to find the loads that the structure were used to carry. It is seen that an exact

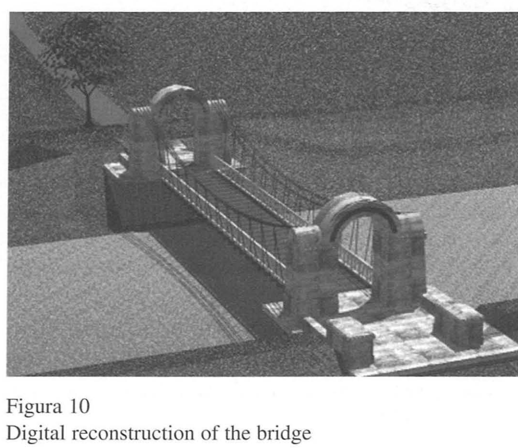

Figura 10
Digital reconstruction of the bridge

reconstruction of the structure as it was before it is impossible. The loads that the bridge should carry today are much higher than the ones used for the earlier construction, even if a pedestrian solution is chosen. As well the rigidity of the structure must be higher in the new construction; this serves in order to reduce the maximum deflection of the bridge. One valid solution can be to change the material of the parapet, originally made in wood, with steel. A steel parapet can have the function of a frame truss with high stiffness. The use of high technologic strength steel can renovate even the cable system.

The analysis of the structure is made, before implementing the Steinman formulas in a spreadsheet, and then comparing the results with data carried out by a finite element program.

The original parapet was made by oak; in the project of reconstruction it is rebuilt with rectangular hollow sections with exactly the same dimensions of the original one. The exact dimensions were founded in historical papers: during renovations of the bridge where presented a detailed report of the work with the dimensions of the renovated elements. The wanted rigidity of the parapet is given by modification of the wall thickness. On the table 1 it is represented a comparison between data of the old structure and the project of reconstruction. Due to the lack in bending stiffeness of the original bridge the maximum deflection was really high. It is possible to reduce this deflection increasing the parapet rigidity. The self-weight of the original structure is much higher respect to the new one because of the greater number of cables, a bigger wooden truss and the presence of a wooden sidewalk.

The biggest problem encountered was because of the river. The actual position of the bridge does not allow a reconstruction: recurrent overflows of water could invest the wooden deck of the structure.

To by pass the problem the masonry piers are taken apart and the foundation is risen to a security level. Than the piers are mount in the same position as they was before.

The cable force on the foundation is separate from the old base. A new footing is made behind the structure capable to absorb alone the cable load. The old foundation will carry only the weight of the bridge and the risen part of the foundation.

Table 1. Bridge Properties, the * values are guessed thanks to historic documents and to other data taken from similar structures of the same age

	Reconstruction	Original
Main span length [m]	34,89	34,89
Sag [m]	2,90	2,90
Sag-to-span ratio	1/12	1/12
Cable area [m^2]	0,01326	0,02851
Cable modulus Ec [N/m^2]	1,62985E + 11	1,2962E + 11*
Truss modulus Et [N/m^2]	2,10000E + 11	1,0000E+10
Moment of Inertia [m^4]	0,00680096	0,00024*
Dead load [KN/m]	11,26	21,86
Live load [KN/m]	18,92	11,28*
Dead load tension H [KN]	591,019	1147,40
Max increase in cable tension due to live load h [KN]	1604,49	1934,22
Max bending moment [KN · m]	494,46	10,89
Deflection v [m]	0,034	0,1536

CONCLUSIONS

Several difficulties were encountered during the reconstruction project. Keeping always in mind the history of the bridge, the original design's decisions and the philosophy that were around the structure it is always possible to find a right solution. A solution that will respect the original structure, and in the meantime can be modern and highly technological, can always be found.

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Wooden masterwork of saline in Ciechocinek, Poland

Waldemar Affelt

The key for understanding of the cultural properties is the set of their associated values. There are many ways of talking about those values: they range from historical to commercial and have tangible or intangible nature. It is a duty of researcher to discover, describe and interpret those values. The presence of certain values should lead to the protection of cultural heritage resource. The crucial question of that concern is an assessment of importance of each discovered value. Any answer causes practical approach to the safeguarding strategy. It is an obligation of cultural heritage managers to preserve those values and pass them to the future generations: this statement applies consideration of sustainable development idea. There are various topologies of cultural heritage values, have been developed since the middle of the nineteenth century. Traditionally that evaluation was associated with an objects of art and monumental architecture. It seems necessary to define methodology of value assessment for the purpose of technical, industrial and engineering heritage. The saline in Ciechocinek combines features of monument of industry and construction engineering. In October 2002 that enterprise celebrated the anniversary of 170 years of production. Basic reference book about the history of Ciechocinek was published by Marian Raczynski, the Chief Director of the Spa (1896–1928), before the WW II. Many of the original archive sources vanished in course of the war and today contemporary authors often have to refer to that book (Raczynski 1935). The method chosen for presentation of the saline in Ciechocinek is based on methodology proposed by the International Centre for the Study of the Preservation and the Restoration of Cultural Property in Rome (Feilden 1993). All pictures were taken by the author.

Local tradition of salt production

Kingdom of Poland was supplied with salt from the royal salt mines in Wieliczka and Bochnia, the places situated not far from the past capital city of Cracow. Situation had changed dramatically after the first partition of Poland in 1772, when the access to those old salt mines was lost. Southern part of the country became incorporated by the Austro-Hungarian Empire. Therefore, the Polish Great Parliament lasting for four years (1788–1792) turn attention to the necessity of searching for the new sources of salt. It was stressed to look for a salty rock rather, than for brine springs. The city of Ciechocinek is situated in the Kujawy region on the left bank of the Vistula river of the two to four kilometres wide proglacial stream valley, at an average altitude of 44 m above sea level. Archaeological traces of manufacturing salt from brine springs run back to the second century BC. The documents of the Roman-Catholic parish in Slonsk - the town once upon the time existing on the area of the present salt works of Ciechocinek being now only known by name —from 13th century mention

production of salt from the local springs. The first bore hole on grounds of private village of Ciechocinek nearby Slonsk was completed in 1791 and gave a brine of very low concentration. For that purpose a drilling equipment was borrowed from Farhwasser in Gdansk. In 1795 Poland had lost independence completely and disappeared from the map. The state of Prussia took a rule over the land of Kujawy and had started drilling in 1798. Until 1801 the brine of 3,8 per cent concentration was obtained. Polish national interest on salt production had had to stop until the Vienna Congress in 1815 when state of Poland so-called the Congress Kingdom was proclaimed. Poland received a certain measure of dependence from the Russia linked by the personal union of vice-king appointed by the Emperor. Poles again approached the exploitation of the salty springs at Kujawy. That challenging project received support of Stanislaw Staszic (1755–1826), pioneer of mining, and the noble Franciszek Ksawery Drucki-Lubecki (1778–1846), Secretary for Treasure and pioneer of industry. Stanis?aw Staszic, scientist, statesman and one of the leading minds of the Polish Enlightenment movement, described a concept of graduation tower, well known after the book on «Gradierwerk» by Janderson published in 1720 in Magdeburg (Iwanowska-Jeske 1983, 61).

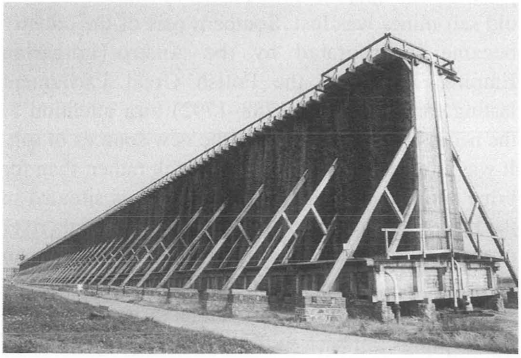

Figure 1
Ciechocinek, Graduation Tower No 3 (listed monument, register No. 424, 1958); state after renovation completed in 1997

CULTURAL VALUES OF SALINE IN CIECHOCINEK

Identity value based on recognition

Construction works had started in 1824 and set of graduation towers No. 1 and No. 2 was completed in 1827. Industrial production of salt in Ciechocinek started on October 21, 1832, after period of trials. The concept of exploring salty water for health treatment purpose emerged as early as in 1827. In 1836 Dr Roman Igntowski, physician, arranged on his own expenses an extension attached to the austere with 4 bathtubs made of copper. In 1842 the Chief Committee for Ciechocinek Health Resort Development was established in Warsaw. Its activity resulted in erection of the new Spa House made of brick with 36 bathtubs in 1847. It was a starting point for making profit from new kind of services —spa business, particularly whereas the profits obtained from the patients were installed successively into the new investments: the Bath House No. I, II, III, and IV with all necessary infrastructure and facilities.

In the middle of the 19. century three parks were laid out, drinking house erected and the Müller's Hotel constructed as timber framed three-storey building. Accommodation was offered by private sector by means of small wooden pensions and mansions. Railways approached the salt works in 1867 passing by the way the passenger station; it was siding 7 km long off the main track from Warsaw to Aleksandórw Kujawski. In course of time the income from salt production became smaller than that one obtained from the spa services. In 1905 the Spa incorporated the Salt Works. Ciechocinek received a status of the town from the Russian authorities, confirmed by the authorities of independent Poland in 1918. In 1932 the hot spa of 37 C was found and the open air swimming pool of dimension 100 × 40 m was constructed between the graduation towers. Its opening ceremony was held with the presence of Prof. Ignacy Moœcicki, the President of the Republic. Interwar period established good reputation of the Spa in Ciechocinek. In 1939 Ciechocnek was named Hermansbaden and converted into the luxury spa for Nazi dignitaries and wounded German soldiers, closed for Poles. After 1945 many investments enriched services and enlarged potential number of patients according to the public health strategy of socialistic state.

Relative artistic value based on research

Saline buildings and structures hardly applies for artistic recognition. The salt works were recently reconstructed, i.e. entire brick masonry and roof trusses were redone following original shape of ca 1890. Directly connected with the history saline are: old engine room made of red brick laid in decorative pattern, ca 1896, recently converted into grocery store (listed monument; register No. 343/A, 1994); brine fountain «Mushroom» in a very heart of the city, designed by W. Noakowski, 1926, supplied form the spring No. 11 drilled in 1909–11. Most prominent in terms of architecture is thermal swimming pool designed by arch. R. Gutt and Eng. A. Szniolis, an example of functionalism, 1932, with changing room capacity for 1500 bathers. On the other hand the image of graduation wall serves as a local logo of curiosity and often arrives on souvenirs. Example of such application is visible at the top of fasade of the New Bath House for Poor, constructed in 1913 (listed monument; register No. 59/496/A, 1964). The city code-of-arm presents a graduation tower, as well. Moreover, numerous buildings of the city posses features of architectural details typical for historicism style: Neogothic Roman-Catholic church according to design by E. Cichocki, 1884; wooden Spa Theatre by arch. Schimmelfennig, 1890; Old Bath House No. 2, arch. J. Majewski, 1898; modernistic building of the Post Office, arch. R. Gutt, 1935; reinforced arch and shell structure of market hall, functionalism, 1938.

Figure 2
Ciechocinek, New Bath House for Poor, facade, 1913; architectural detail made of plaster depicting the scheme of graduation tower

Relative historic-technical value based on research

The whole saline project was developed by the Konstanty Wolicki, the pioneer of Polish Mining. He bought a land from settlers in villages of S?onsk, Siarzewo and Wo?uszewo and incorporated those lots to the state owned grounds. Structural drawings were supplied by Eng. Jacob Graff, professor of Mining Academy in Kielce. Parallel graduation towers No. 1 and No. 2 were executed by builder K. Knake and completed in 1827. Graduation Tower No. 3 was erected in 1859. The table below shows a basic data of graduation towers (Table 1)

Those towers are built of thick logs supported by wooden blocks sunk deep into the ground. The brine tanks are elevated 120 cm above the ground. The tank structure serves as a base for frames of the graduation tower. Posts are made of long timber, thousands of several meters long pine trunks were delivered on site from the state Kampinos forest and private owners (Tloczek 1958). That entirely wooden framing

Table 1

Graduation Tower	Length [m]	Height [m]	Width [m]	Tank volume [m^3]	Contents of brine [%]
No 1	648	14	9	5.800	up to 10
No 2	719	16	9	6.300	20–22
No 3	333	12	9	2.900	up to 16

system, narrower at the top, is filled with horizontal layers of blackthorn sheaves. Those sheaves are spare elements and they have to be changed every 5–10 years. For that purpose a plantation of blackthorn was layout nearby. Several meters over the ground on the top there is a gallery along which run wooden channels with brine pumped from the spring. From beginning there were discussions whether it should be powered by wind mill or modern steam engine, thus various solutions were applied including both media at the same time. While the brine drops down through the blackthorns then the wind increases a process of water evaporation, and oxygen in the air changes into ozone due to the sun radiation. Along that permanent process a healing aerosol rich in iodine is produced particularly intensively on sunny and soft-windy days.

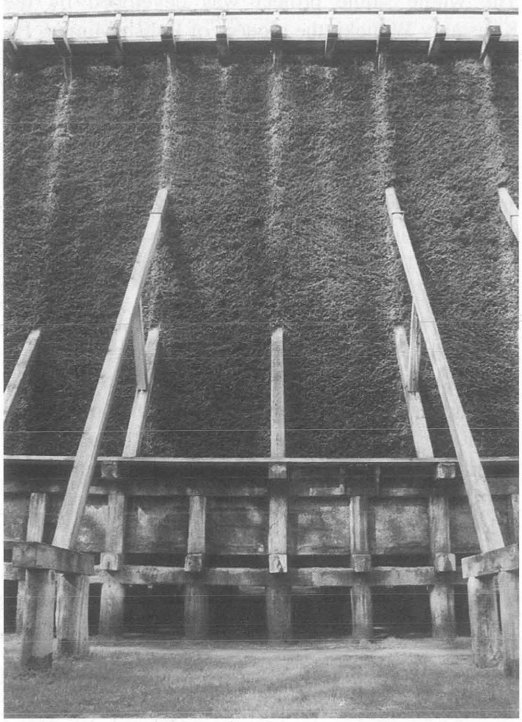

Figure 3
Graduation Tower No. 1 (listed monument, register No 424, 1958). In front view are seen the major structural items: founding blocks, main and girt beams supporting the tank, inclined braces, wall of blackthorn, top gallery with railing

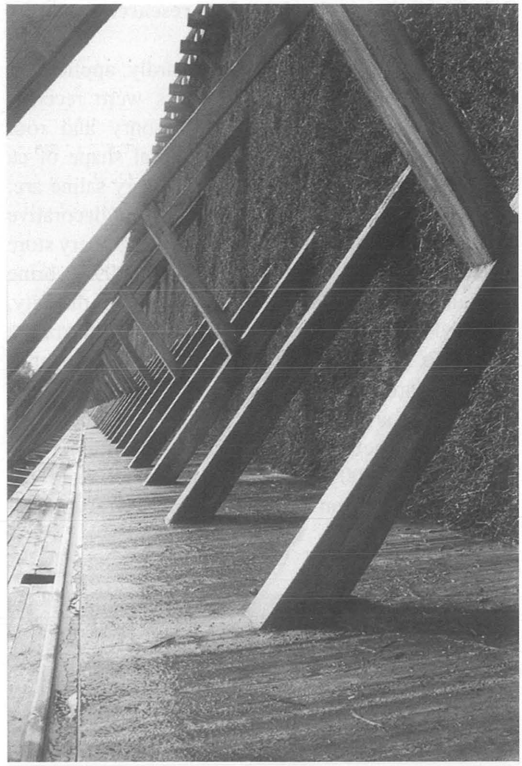

Figure 4
Graduation Tower No. 1. Side aisle view: tank cover-floor collecting brine drops blown by the wind, drain channel, maintenance deck with inspection hole

Due to that phenomena the temperature near the graduation walls is lower and humidity is higher that remains a sea-like climate enriched with brine-iodine-bromine-ozone particles. That aerosol heals slowly respiratory sufferings and improves results of many other treatments. The brine percolates down the tower's wall to the bottom tanks, increases the salt concentration from natural initial 4,5 to 22 per cent, and then it flows through a pipe to the salt works. It was experimentally discovered, that if concentration exceed over 22 per cent then the salt crystals are developing on the blackthorn walls, and the gust can remove them. Circulation of brine is operated by master of graduation wall called «graduator» whose decisions are based on assessment of the current weather conditions and the brine concentration

measured by means of salt aerometer. After relevant number of cycles the brine reaches concentration around 20 per cent and becomes ready to be transferred for the further treatment in salt works located around 2 kilometres to the east. In the past the piping system was made of wood, and example of such pipe is exhibited on site. Brine is collected in two sedimentation tanks made of wood around 1840. They are situated 2 meters above the ground on wooden posts and they reach high of 8 m; their length and width are 59,6 ¥ 10,75 m and 15,6 ¥ 8,8 m, respectively. Such arrangement protects a brine in case of flood, and creates an opportunity for gravity inflow of brine to the preheating tanks inside of salt works building.

Winds in Ciechocinek are blowing mainly from the west and south-west, thus salt works were located on distance from graduation towers in order to avoid brine pollution caused by smoke. Another reason was to move the plant near to the Vistula river for the purpose of transportation. A canal was dug in the old river bed between production yard and the river bank. After railway access to Ciechocinek that solution became abandoned. Salt works were equipped with the sets of preheating tank and pan made of riveted iron sheets sealed with special lime putty that harden while heating. After destruction of that site by flood in February 1871 the new building for salt works was erected on the same lot but about 200 meters to the south, and old pans were reused. At the present the rectangular preheating tank is 10,7 m long, 7,4 m wide and 1 m deep; here concentrated brine is heated up to 45 C. After reaching that temperature the brine

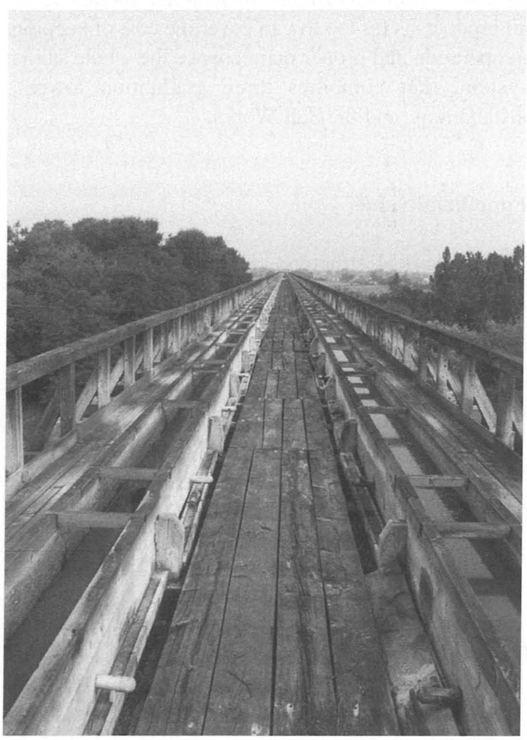

Figure 5
Graduation Tower No. 1. Top gallery with two channels supplying the brine; in the middle maintenance deck

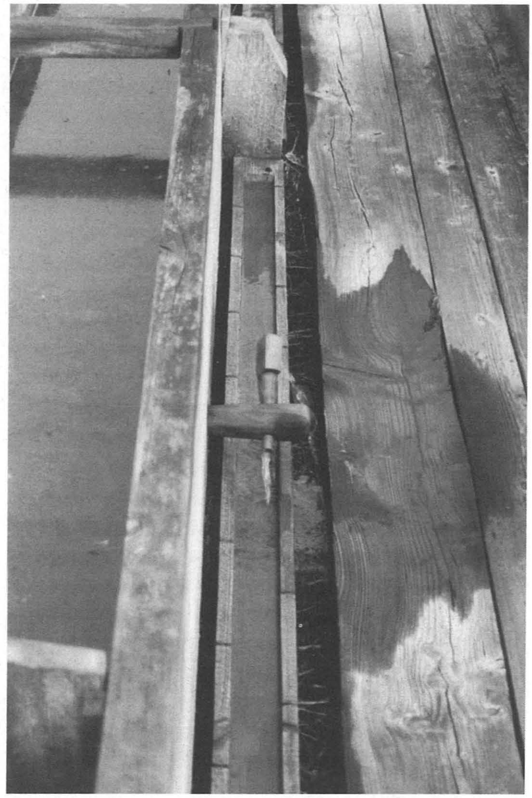

Figure 6
Graduation Tower No. 1. Top gallery; detail of brine valve made of chestnut wood, distributing the brine into secondary channel, where from brine drops directly onto blackthorn filling

is pumped to the pan 14,5 m long, 8,0 m wide, and 0,6 m deep. It takes one day to rise a temperature up to 104 C and to form the salt crystals. The whole technological process is conducted by the «cooker» —the man who supervises preheating tanks, fire pans, salt crystals collecting, drying and their transfer to salt depot. As well, he menages the production of the side-products: slime and bathing lye. Two chimneys of the Salt Works became a dominant of the landscape being visible over earthen works intensively developed in order to protect fields and settlements against the flood. Area of the spa including Graduation Towers is surrounded by ramparts 6,4 km long in total. Since the 18. century many holes were drilled, and among them the seven are exploited now; the deepest source runs from the depth of 1450 m.

Rarity value based on statistics

In the 19th century several projects of graduation towers were executed in area of Poland, but only the enterprise in Ciechocinek lasts for so long. However the profit from salt production had lowered successively and in 1853 brought a deficit, the plant was kept into operation. It may be seen as an action against the capitalistic rules, but from historical point of view it is a marvellous example of cultural added value enhancement. Development of the spa business was based on wise maintaining the public image of miraculously health giving Graduation Tower. At the present the saline in Ciechocinek is only in Poland example of industrial plant from the first half on the 19th century has been still in use.

Figure 7
Graduation Tower No. 3. The picturesque windmill for powering the pump illustrates past appearance of the structure

CONTEMPORARY SOCIO-ECONOMIC VALUES OF SALINE IN CIECHOCINEK

Economic value

There is modest demand for saline products: salt, bathing lye and slime. The Graduation Towers operate in small percent of their productivity. From one technological portion of concentrated brine of 80 cubic meters —equal to the pan capacity— it is produced 240 kg of salt. Forecasted Saline efficiency assessed in the thirties was 10.000 tons per year. At the present the salt production did not exceeds one thousand tons. Half of the historical installations of the Salt Works was already dismantled. Income from all products is far too low to cover the cost of keeping in operation and proper maintenance the whole saline system, that combines three graduation towers, installations, and the Salt Works.

Functional value

The geological structure of Ciechocinek is known from around sixty holes drilled from 18. century with the deepest one of 1825 m. Today nine sources are in use including three artesian warm brine springs of 37 C, called thermals. The Ciechocinek waters are composed of chloride, sodium, bromine, iodide, and iron. They serve as a raw material for the production of salt, mud, medical lye, and table mineral waters marked as «Krystynka» and «Kujawianka». According to present tendency of market demands, the sector of leisure, rehabilitation and health improvement seems to be one of the leading streams of services and job demands. Under those circumstances the potential of Ciechocinek is promising: valuable brine natural products, clean air free from industrial and urban pollution, parks and green areas well arranged for walking, places of natural beauty and cultural monuments in neighbourhood environment. Moreover, the thermal swimming pool is an attraction for the families from

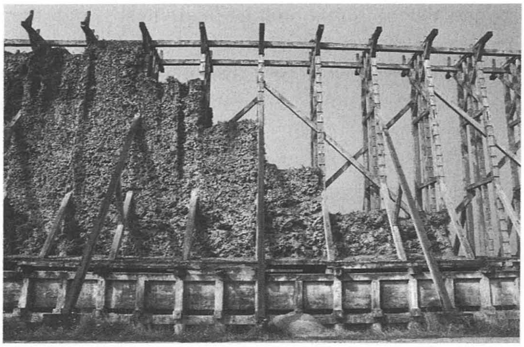

Figure 8
Graduation Tower No. 2 (listed monument, register No. 424, 1958). Dramatically dismantled blackthorn filling has open an eye-view access to authentic wooden structure of 1827, unfortunately being exposed directly for weathering and deterioration

the whole region as a place of summer leisure; the installation of thermal fountain cascade-like is exploit intensively by visitors as a natural massage.

Educational value

Well preserved structure of graduation towers made entirely of wood is an unique example of long lasting performance of that structural material has been explored properly under certain self-preserving conditions, i.e. being permanently moisten with brine. It may serve as evidence of the past building craftsmanship and traditional art of the carpentry. The Ciechocinek saline is rather unknown and rare example of industrial investment developed by the state in the first half of the 19th century (Gerko 1998). That contributes a lesson on national economical history and the first faze of the Polish industrialisation. Spatial progress can be studied on site, and the town fabric development from the nucleus point of the Graduation Towers up to the modern health resort facilities may be observed.

Social value

The Graduation Towers and the Salt Works have been kept in operation continuously for 170 years. That emerged professions, tools and know-how being expressed by certain words, names, and procedures. Oral tradition of that specific craft is not recorded and can be abolished very easy. More than 200 workers and foremen were employed in saline around the middle of the 19th century; records from the great flood in 1871 say about 170 workers who lost a job due to distraction of Salt Works caused by water; at the beginning of the 20th century less than 50 workers were employed; before the WW II saline employed up to 82 men (Gerko 2001). Majority of the Ciechocinek families was and is related to the spa services and receives income thanks to visitors coming here for few weeks of treatment or just as one-day tourists attracted by the curiosity —enormous in

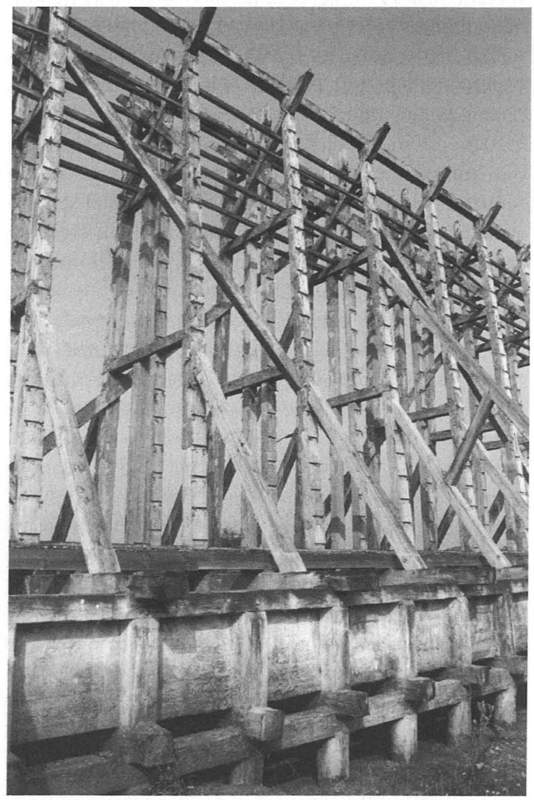

Figure 9
Graduation Tower No. 2. Wooden superstructure previously hidden under blackthorn filling; now it let us appreciate the ingenious concept of the past engineers and high quality of craftsmanship

size Graduation Towers seen in operation. To walk along those permanently wet blackthorn walls was a symbol of middle class habit during the real socialism period, and even a popular humorous song was devoted to that behaviour. In our days both the clients and the spa offer have been changed. There is no visible evidences of mass interest any more. New private development of housing estate and luxury beauty clinics is addressed to nourishes.

Political value

On the land of Poland there is no similar to Ciechocinek example of long lasting industrial production and related spatial development. Due to specific history of that country there is no way to omit the relations with Germany, Russia, and their citizens. Additionally, a role played by Jewish population should be taken into account. Taking about European dimension recalls traces of international co-operation in the past. In the 17th century Mennonites came to that area and settled in Slonsk-neighbourhood of the Salt Works site; document of 1776 issued by the Polish king Stanislav August confirms their right. In spring of 1798 Alexander von Humboldt (1769–1859), mine inspector, came to Ciechocinek as scientific consultant. He assessed positively the discovered brine sources and recommended their further exploitation (Gerko 1995, 155). During the Napoleon period the fields with brine springs were given to the French marshal Nicole Jean Soult (1769–1851), Duke of Dalmacia. He tried to establish a private monopoly for salt production by means of restriction addressed to local population. It was prohibited to «cook» salt within a radius of 4 km counted from the church in Slonsk-standing there from the 15th century; that lot exactly is occupied by the present Salt Works. Present saline project emerged under auspices of the Russian Emperor Alexander I, and for its execution the Polish State hired Eng. Jacob Graff (1780–1854), alumnus of Bergbau Academy in Freiburg, Germany, professor of Mining Academy in Kielce, Poland. Another engineers and the state officers of German origin involved in saline development were: Becker, Hann, Englert, Rost, Stark, Ullmann, and others. From the very beginning the saline was insured at the English Insurance Society «Alliance». The State Bank ownership of saline expired in 1870, and then Ministry of Revenues in Moscow took a rule. From 1887 the saline went on lease to hands of Russian general Boris Glinka-Mawrinow, who established a stockholding in 1890. In that time the present building of Salt Works was erected and equipped with four sets of preheating tank and cooking pan.

PRESENT CHALLENGES

Today the technological system of saline consists of: spring No. 11, drilled in 1909–11, source of 4,55 % brine; set of three Graduation Towers of total length 1741,5 m; the Old Pomp Station located between the Graduation Towers (listed monument, register No. 322/A, 1993); main and secondary pipe system for brine transportation; building of Salt Works of 1890 with two sets of pan and preheating tank; two reservoirs for brine impurities sedimentation, capacity of 823 m^3 and 3011 m^3 entirely made of wood ca. 1840 (listed monument, register No. 424, 1958). Local people are tied emotionally to the Graduation Towers —a famous landmark of Ciechocinek. Motive of that structure often appears on local souvenirs. However there is certain number of unemployed, and services connected to health recovery and rehabilitation activities are almost the only possibility for fulfilling the jobs desire. Unfortunately, the most accessible for general public element of the complex-thermal swimming pool has been closed for summer season of 2002. Older generation keeps in mind images of the flourishing spa resort from several years ago. Author of this paper has undertook promotional measures for the saline-a prominent monument of industry and engineering: the Saline in Ciechocinek is proposed as an entry in the atlas of structural and civil engineering monuments of the «Visegrad Four» States published by the national professional chambers and societies of building engineers in Poland, Slovakia, Czech Republic and Hungary. It is expected to have it printed in a middle of 2003. Moreover the lecture about cultural values of the Ciechocinek Saline was given by the author during the 5th Forum of Conservators in Torun, Poland, February 2002, and the poster on «Salinepolis of Ciechocinek» was presented during the European Union Conference on Cultural Heritage Conservation, Cracow, Poland,

May 2002. The rarity of wooden structure of graduation towers is doubtless within the country, however the world wide recognition would be much helpful for their strategic safeguarding and prospect of application to the World Heritage List. In terms of industrial heritage the whole group of saline buildings, structures and installations is an unique example of early industrial site has been in operation for 170 years until now.

Conclusion

The city of Ciechocinek has grown from saline on exploitation of brine sources. The features of related technology and healing treatments have shaped the landscape through the years. In nowadays Ciechocinek is still the biggest health resort of Northern Poland. The Spa Company owns numerous sanatoria and the saline —monument of brine mining with immense masterwork of carpentry— the Graduation Towers, that operates in the same manner as it was at the very beginning. To keep up reputation of the spa in Ciechocinek means to conserve its heritage and to keep on going saline-alive monument of industry. This is only way to pass its cultural values to the future generations.

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Architecture as talisman: The hidden links between Vitruvius' theatre and Palladio's villa «Rotonda»

Alessio Ageno
Maura Frilli

Giulio Camillo Delminio, one of the most renowned men of the first half of the XVI century in Venice, became famous all over Europe for having constructed and actually built a wooden theatre with the purpose of improving the power of memory. This theatre was full of various «images of memory», such as «interior talismans», well illustrated by Frances Yates in her beautiful book *The Art of Memory*.

Following the magic treatise *Picatrix*, the talisman becomes an object into which one infuses life, not only by means of magic rituals, but also, as in the ancient example of Egyptian sculpture, through the meticulous application of exact geometrical and mathematical proportions.

Thus, the wooden theatre, into which the images of the memory were placed, was itself a talisman; built following the proportions of Vitruvius' theatre, it was a real and proper talisman of the world, a «building» with such a shape and geometric relationships that the magnificence of the whole life of the Universe was infused into it.

Vitruvius' theatre, in fact, created a vast amount of interest in Venice throughout the XVI century, starting with Alvise Cornaro and Falconetto to Barbaro and Palladio, and culminated in the Palladian project for the «Teatro Olimpico» at Vicenza.

In this paper we present an analysis of Palladio's masterpiece villa «Rotonda», starting from the basis of an original investigation regarding the significance of the Vitruvian theatre. This analysis is based upon other classical authors, such as Ptolemy, with the aim of clarifying the most profound analogies one encounters in the «De Architectura»: astronomical and astrological, mathematical, geometrical and musical analogies.

Surprisingly enough investigation regarding the profound significance of geometry emanating from the Vitruvian theatre provides us wth a key in finally explaining —adopting a unitary method— and solving the enigma of the whole system of dimensions and reciprocal ratios among the various rooms comprising the villa, including the circular central room.

Moreover this study provides us with other meanings in addition to those concerning the single building considered. Thus it is actually proved that the same musical basis of proportions sets both the ratios between the dimensions of small integer numbers (which are fundamental in musical consonances) and the geometries of the regular polygons inscribed in a circle, «modulated» by the number of sides.

Finally, an interpretation is proposed which surpasses the denial of importance of irrational ratios in the architectural proportions of the Renaissance. This opinion was a principal thesis in the fundamental essay *Architectural Principles in the Age of Humanism* by the great historian and critic Rudolf Wittkower. Moreover, the two aspects indicated by the same Wittkower, leading architectural theory of

the Renaissance, are now unified. In fact the common musical analogy appears to be the same root of circular geometries and simple proportions.

Vitruvius' analogical theatre

Of the twelve chapters contained by the fifth book of the Vitruvius' *De Architectura*, dedicated to public places and buildings, at least seven (from 3 to 9) are dedicated to the theatre; among these, two deal with musical themes for the most part, while the remainder make constant references to this subject. The particular attention given to the theatre is justified by its being a perfect example of the concept which Vitruvius has of architecture. According to this conception a building reaches perfection when it appears to be *epiphany* of nature, in a signification to which contribute the sphere of knowledge, together with that of religious sentiment. As an instrument of knowledge throughout the world, in addition to its position occupied in that of man, construction permits the understanding and representation of nature.

Vitruvius demonstrates his own agreement with the tradition of expressing the concept of an idea, which is not just beautiful but perfect. Therefore, both beautiful (*venustas*) and true (*firmitas* and *utilitas*). On these grounds, the importance of *ratiocinatio,* and paying tribute to the ancient scholars and scientists who made it their task to hand down their discoveries in writing, is understandable.

In this context the theatre becomes a place where the nature of sound is revealed: of a physical and sensorial nature together with a cosmic and metaphysical nature. The established rules concerning the inclination angles of the cavea, the height of the walls of the *praecinctiones* and the acoustic instruments (*echeia*) apparatus situate the sensitive nature of sound inwards with regard to the architectural epiphany of the cosmic nature of sound, which generates the formative geometry of the theatre. The mathematical and musical *ratio* regards both the overall geometry of the building and the arrangement of the acoustic instruments (*echeia*) whose value consists in its naturalness: «Ergo veteres architecti naturae vestigia persecuti indagationibus vocis scandentis theatrorum perfecerunt gradationes, et quaesierunt per canonicam mathematicorum et musicam rationem, ut, quaecumque vox esset in scaena, clarior et suavior ad spectatorum perveniret aures» (Vitruvius, V. 3).

That the nature of harmonies, which regulate the theatre plan, is precisely cosmic is confirmed in the ninth book (Vitruvius, IX. 1) where the description concerning the division of the sun rays from one zodiacal sign to the others concords with the *trigonals* (triangles) which form the geometric base of the plan for a Roman theatre. That it is metaphysical is proven, once again in the ninth book, by the invocation of a divine intellect to explain the structure of the Universe (Vitruvius, IX. 11). If the *echeia* and the twelve zodiacal points, which dissect the theatre plan, respectively reflect the physical and metaphysical aspect of the sound, their chord is total when we consider that it is the same range of musical notes (the perfect system constituted by two octaves) which covers the *echeia* apparatus and the zodiacal circle articulated according to the twelve tones upon which the Greek perfect musical system is extended.

Only in a building considered as a complete epiphany of sound is it possible for man to definitively take his place, with his poetry and song, so that this poetry and song enter into complete resonance with the theatre space, make it vibrate with its own life. But as this space also represents the figure of the cosmic nature of sound, of the musical structure of the world, man, defining and being in tune with its own sonorous nature, the voice and the song, also defines his own position in the world.

This is the sense of the imaginative synthesis focusing upon the comparison between theatre and the musical instrument —in this case an instrument of expression rather than one of knowledge: «Uti enim organa aeneis lamminis aut corneis echeiois ad chordarum sonitum claritatem perficiuntur, sic theatrorum per harmonicen ad augendam vocem ratiocinationes ab antiquis sunt constitutae» (Vitruvius, V. 3).

The harmonies of the circle

The plan of the Roman type of theatre provides for the inscription, in a circle, of four equilateral triangles, so that the apex lies upon the circumference at equal distances, one from the other (Figure 1).

As a comment regarding geometric construction it is stated: « . . . quibus [scil. trigonis] etiam in duodecim

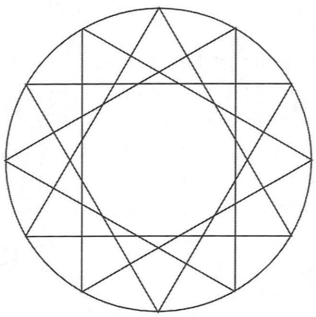

Figure 1
Generative plan of the Roman theatre according to Vitruvius.

signorum caelestium astrologi ex musica convenientia astrorum ratiocinantur» (Vitruvius, V. 6).

This brief passage should be considered in relation to that disclosed in I. 1, where it is stated: «Similiter cum astrologis et musicis est disputatio communis de sympathia stellarum et symphoniarum, in quadratis et trigonis, diatessaron et diapente, . . . » (Vitruvius, I. 1).

There are numerous ideas and problems which refer to these few lines of *De Architectura;* almost a glimmer opening upon a vast and articolated galaxy of thoughts and conceptions, which represented a common patrimony for various cultures, despite diverging differences, both in time and space.

The first matter to be clarified is the nature of the relationship between the geometric constructions of the theatre, the zodiacal constellations and the musical intervals. The comparison between the first two points is comparatively simple: the most evident analogy is that the theatre plan makes room for twelve apexes —the number of the zodiacal signs. In order to continue we require the assistance of an ancient astrological source: the complete affinity between the two diagrams can be found referring to Ptolemy's *Tetrabiblos*, a work dedicated to the astrological predictions of the great astronomer of the II Sec. A.D. Although post-Vitruvius, this work amounts to the total sum of ancient astrology and contains, in systematic form, the elements of the «immense . . . literature which, throughout the centuries, from the beginnings of Hellenism to the decline of Greek-Roman antiquity, have assisted the cause of astrology» (Boll, Bezold and Gundel 1977, 39).

Chapter I. 14 of Ptolemy's *Tetrabiblos* is dedicated to the aspects of zodiacal signs (*radiationes, suschematizomena* or *schematismoi*), i.e. to one of the various types of affinity or reciprocal relationships between the signs. In order to understand how these aspects arise, we must consider the twelve constellations equidistant on a circumference. The simplest aspect is the opposition, in other words two signs connected by a diameter; the others are the trigonal, three signs to three apexes of an equilateral triangle (Figure 2); the tetragonal, four signs on four apexes on a square (Figure 3); the sextile, six signs on the apexes of a hexagon (Figure 4).

It is evident that the geometric plan of the Roman theatre is the same as that of the trigonals.

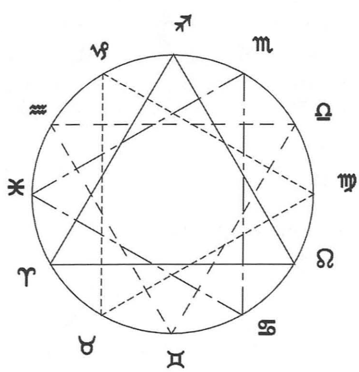

Figure 2
The trigonals

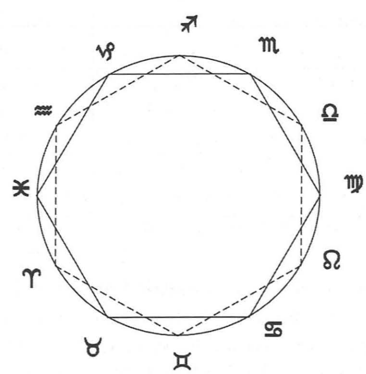

Figure 3
The tetragonals

The central point is the relationship between this geometry and the musical intervals. For this reason it is necessary to illustrate the third point of the analogical series made up of, respectively, polygons, inscribed upon the circle, astrological aspects and musical intervals. It is Ptolemy who, once again, provides us with an exhaustive and elegant theorisation in his treatise on harmony (Ptolomaeus *Αρμονικων Βιβλια Γ*, III. 9), in which he compares consonances to the relationship between the zodiacal signs. In order to establish the comparison the circle is divided into twelve equal parts which, for our purposes, we will refer to as degrees —as twelve is the common minimum multiple of three and four. The small whole numbers express the harmony intervals. The zodiacal aspects are the opposition, the trigonal, the tetragonal and the sextile, corresponding, respectively, to the diameter, the triangle, the square and the hexagon: to clarify all of this the diagram below (Figure 5) demonstrates the figures relative to the aspects.

The Table 1 refers to the above diagram as a brief indication of how many degrees each arch, which sub-tends the sides of the polygons, measures: more precisely, the four letters (ABCD) indicate the whole circle, the three letters indicate arches greater than 180 degrees and AB indicates the semi-circle, while the two letter combinations indicate the arches corresponding to only one side:

Table 1

The whole circle ABCD	12
ABD	9
ABC	8
AB	6
AC	4
AD	3

From the numbers indicated above it is possible deduce all the relationships which produce the harmonies as classified by Ptolemy.

In addition to the real and proper harmonies it is interesting to note how the tone is also integrated corresponding to the 9/8 relationship.

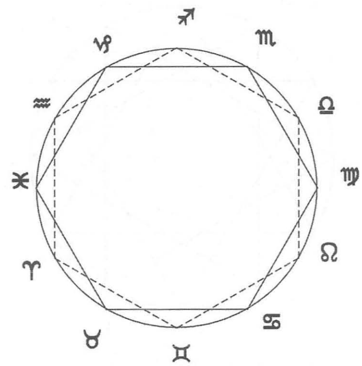

Figure 4
The sextiles

As can be seen, this treatment provides us with an insight into a kind of musical mathematics with regard to the circle, where deeper investigation of the relationships between intervals and geometrics appears to go beyond purely astrological objectives. However, after general examination Ptolemy returns to the aspects, in order to explain the same correspondence between the triangles and the fifths and between the squares and fourths, precisely referred to by Vitruvius (« . . . in quadratis et trigonis, diatessaron et diapente . . . »):

Therefore, according to the same classifications [those referred to in the table above], the fifth corresponds to the

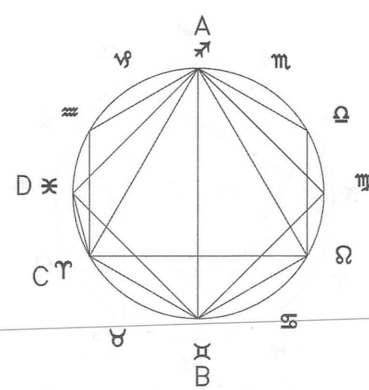

Figure 5
The geometry of the aspects

triangle, the fourth to the square and the tone to the twelfth part of the circle. In fact the ratio between the circle and the semi-circle AB is the double ratio [1:2, octave], the ratio between the semi-circle and the arch sub-tended at the side AC of the triangle is the 2:3 ratio [fifth], whereas the ratio between the same arch sub-tended at the side of the triangle and the arch sub-tended at the side AD of the square is the 3:4ratio [the fourth]. (Ptolomaeus *Αρμονικων Βιβλια Γ*, III. 9)

All things considered, the triangle corresponds to the fifth as it is in this relationship with the semi-circle which, in turn, is at the eighth with the circle, whereas the square corresponds to the fourth because it is in this relationship with the triangle. Vitruvio bears witness, then, to the antiquity of these relationships which date back to at least a few centuries before Ptolemy.

The significance of the division of the circle is declared by Ptolemy using these words: «Nature, therefore, divided the zodiac into twelve according to reason, as it also formed the two octaves of the perfect system of twelve tones and the tone closest to the twelfth part of the circle» (Ptolomaeus *Αρμονικων Βιβλια Γ*, III. 9).

Therefore, the double octave contains twelve tones, and the tone can be compared to the side of the dodecagon. The division of the double octave into twelve throws light on the true nature of the musical analogy of the circle. Still referring to the *Harmonica* (Ptolomaeus *Αρμονικων Βιβλια Γ*, III. 8), the twelve signs of the zodiac are, in fact, placed, first, along a line representing the two octaves, with Libra in the centre corresponding to the «*mese*» musical tone (first octave) and with Aries corresponding to the two outermost musical tones, the longest and the highest (Figure 6).

Then the two extremes of this line coincide, generating a circle: the zodiac circle reproduced related to the fifteen notes of the Greek perfect musical system, corresponding to the twelve tones of the double octave (Figure 7).

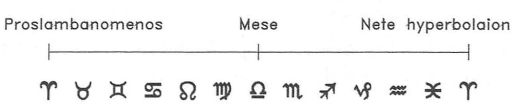

Figure 6

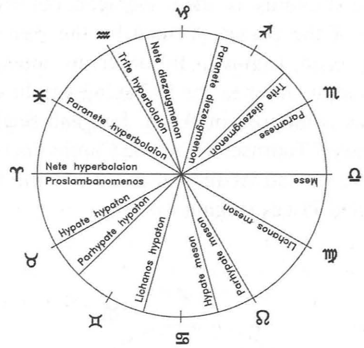

Figure 7

The nature of the analogy is, therefore, evident: the circle is no more than a Pythagorean cord, whose extremes coincide, the divisions of the circle are equivalent to the partitions of the monochord and these generate the same relationships.

After all, as Barbaro states in his comments to Vitruvio, «the arithmetic rules, then, are those which create the unity between music and astrology, due to a common proportion» (Barbaro 1584, 24).

The nature of the relationship between geometry and sounds provides us, therefore, with a basic mathematical aspect but, in addition to this, and perhaps exactly for this reason, there are also other analogies to be considered; this comes as no surprise once touched upon by the astrological environment, a type of speculation which, as Warburg said, «allies . . . two spiritual powers, completely heterogeneous . . . mathematics . . . and the power of demons» (Warburg 1966, 331–332).

Another interesting implication of the astrological trigonals regards the link between these, the cardinal points and the winds. This aspect concurs, as will be seen, with a harmonic and complete interpretation of the analogies decipherable in the plan of Palladio's «Villa Rotonda».

Ptolemy, in the *Τετραβιβλος*, classifies trigonals according to their direction. On the assumption of this classification they are the domiciles of the planets in their various signs. In fact this coupling of trigonals and cardinal points should concord with the traditional identification of Jupiter with the north, Venus with the south, Saturn with the east and Mars with the west. The identification of the trigonals with

the cardinal points is then averaged out from the domicile of the planets formed by the signs which compose each trigonal. In order to identify the trigonals let us refer to the following graph: the first trigonal is composed of Aries, Leo and Sagittarius, the second by Tauruse, Virgo and Capricorn, the third Gemini, Libra and Acquarius and the fourth Cancer, Scorpio and Pisces (Figure 8).

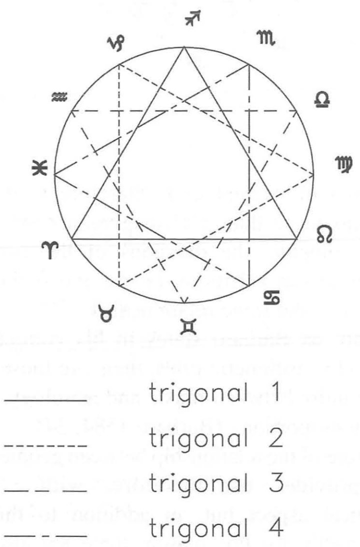

Figure 8

In order to understand Ptolemy's thinking we should follow what he says concerning the first trigonal: «This trigonal is prevalently north orientated for the governing part concerning Jupiter, which is prolific and blustery like the north wind. To be also domiciled with Mars the trigonal suffers a combination of western winds . . . » (Ptolomaeus *Tetrabiblos*, I. 19) Therefore, it is the affinity among the planets, together with the various winds, which determine the direction. Essentially the wind directions are the same of those of the cardinal points. The directions which Ptolemy associates with the four trigonals are demonstrated below (Figure 9).

It is natural, at this point to make a comparison between these additional analogies, which originate from the same background as the musical ones, and

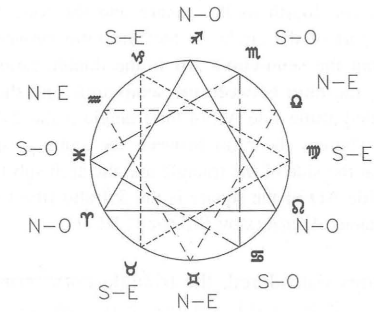

Figure 9

the urbanistic precepts provided by Vitruvius in the first book, on the basis of which an extensive and extremely detailed theory of the winds is expressed. This comparison acquires even more importance if we consider a passage contained in the ninth book of *De Architectura.* Expounding the variations concerning the sun's course with regard to the succession of the zodiacal signs, Vitruvius mentions, while speaking of the Pisces constellation, the name of a wind: «Ab aquarium cum ingressus est in pisces favonio flante, scorpionis comparat aequalem cursum» (Vitruvius *De Architectura*, IX. 3).

The reference to the name of a wind is particularly significant: in a context, which to our modern thinking appears to be completely alien, the emphasis upon «favonio flante» (west wind) is evidence of how natural astronomical references were during Vitrivius' epoch. The Favonius, equivalent to the Greek ζεφυρος, is the westerly wind linked to Pisces, not through the domicile planet Jupiter (north) but through the affinity of the trigonal, as Mars (west) has its domicile in Scorpio, which belongs, as does Pisces, to the fourth trigonal. This brief indication makes it possible to discern a unitary horizon, which passes from the first book of *De Architectura* (the urbanistic indications, cardinal points and winds) to the ninth book (the sun, which radiates light throughout the Universe following triangular geometries and, again, the winds and cardinal points related to the zodiac), passing to the fifth book (the geometrical constructions of theatres and their astrological and musical connections).

VITRUVIUS' THEATRE AND PALLADIO'S VILLA «ROTONDA»

The theatre, in particular the renewal of the old theatre described by Vitruvius, was considered to be a characteristic trait of Venetian Renaissance culture by the great historian Frances Yates. It is interesting that a historian of Culture should make such a declaration rather than a «simple» architectural historian. The reason for this is the coincidence of the motives which Venetian culture borrows from the previous Florentine elaboration of Marsilio Ficino and Pico della Mirandola, with significant extracts from the Vitruvian treatment of the theatre.

This centrality of the Vitruvian theatre has also been examined in modern architectural historiography: the Vitruvian matrix of the Odeo Cornaro, designed by Faconetto for Alvise Cornaro, was, in fact, examined in addition to that of the «Teatro Olimpico» by Palladio (Magagnato 1992). The most profound influence of the Vitruvian theatre upon Venetian Renaissance is, however, perhaps, that indicated by Frances Yates, i.e., that which emanated from the Theatre of Memory by Giulio Camillo Delminio. This theatre, which was also actually constructed, was conceived as a mnemotechnical support in a philosophical and magical (in Renaissance terms) sense.

This theatre was full of various «images of memory», such as «interior talismans», well illustrated by Frances Yates in her beautiful book *The Art of Memory* (Yates 1966).

Following the magic treatise *Picatrix*, the talisman becomes an object into which one infuses life, not only by means of magic rituals, but also, as in the ancient example of Egyptian sculpture, through the meticulous application of exact geometrical and mathematical proportions.

Thus, the wooden theatre, into which the images of the memory were placed, was itself a talisman; built following the proportions of Vitruvius' theatre, it was a real and proper talisman of the world, a «building» with such a shape and geometric relationships that the magnificence of the whole life of the Universe was infused into it.

The point to be considered as essential in the creation of a talisman is, therefore, its proportional structure. The study of proportions referred to in the designs illustrating the Four Books by Palladio was effectively a fundamental guide for his exegetists, from Briseux to Wittkower.

In the extremely important modern essay regarding Palladio, *Architectural Priciples in the age of Humanism*, by Rudolf Wittkower, many of Palladio's villas are analysed interpreting the relationship between the dimensions of the rooms in light of the proportional means of various type, both essentially musical or simply geometrical or arithmetical, but referring exclusively to rational relationships between small integer numbers, which represent the length of the various dimensions on the plan of the rooms which make up the building.

For instance, in the overall proportion of Villa Barbaro at Maser, the fourteen assumes a coherent role if one interprets it, according to Wittkower, as the arithmetic mean of two other dimensions measured from the numbers 12 and 16. For this derivation, however, there is not a direct musical analogy. Nevertheless, the proportions of the Maser villa referred to in the *Quattro Libri,* are presented, in this light, as a perfectly co-ordinated joint combination. There exist, however, in the Palladian treatment, other plans where dimensions of fourteen feet are not compared to the other two terms of the proportional arithmetic ternary. We could, therefore, consider a repertory of measurements which spans the entirety of Palladio's projects, as if a sole proportional co-ordinated the architect's entire collection of works and where the explanation of each particular proportion lies. If, on an ideal level, this assessment is ultimately close to the truth, it is, in reality, both laborious and futile to draw conclusions with regard to these particular applications as consequences of a theorem. For the dimension of fourteen feet it is possible to note how, in more than one case, it is close to the dimension of 20 feet. This causes one to think of an implied relationship of the diagonal to the side of the square. This relationship is, moreover, among the seven suggested in the first of the *Quattro Libri* for the measuring of the rooms.

The impression could arise, then, that certain relationships, even though numerically incomensurate, are, nevertheless, the result of simple operations, as Palladio himself preferred (in the First Book the simplicity is exalted) and, further, that these themselves express the will of harmonic creation.

If it is possible to examine the nature —in the true numerical and geometrical sense— of the

proportional which provides the measurement of fourteen feet (as well as that of thirteen and seventeen) in many designs of the Palladian treatise, it is necessary to completely dispense with the numerical commensurability and the dilemma between geometry and numbers if we wish to present an interpretation which consolidates, into one unique description, the harmonic-musical analogy and significance which come into evidence during the critical acclaim bestowed upon this monument: Villa Almerigo, referred to as «La Rotonda».

In more general terms of the accepted evaluation we can cite two references. Using the words of Camillo Semenzato: «The Rotonda . . . can be considered as the most emblematic expression» of Palladio's art, «it is an exemplary paradigmatical building, so as to combine all the fundamental aspects of the Palladian problem». On the other hand he also emphasises the «exceptional ch aracter, which, under all aspects, is exactly that of the Rotonda».

The exceptionality is in its type: «The Rotonda escapes . . . completely from the classical typology of the farmhouse-villa». Palladio does not place it, in the *Quattro Libri*, in the category of the villas but in that of the palaces.

Therefore, the critical position is that between the emblematical and the exceptional and this double reference appears, however, to be contradictory. The exceptionality, then, is not only present in the typological disparity among the most frequented of country and suburban villas; the geometry and organisation are exceptional. At first the various numbers appear to be irreducibly irregular which, in the design referred to in the treatise, determine the proportions and reciprocal relationships of the rooms. Only a few of these can, in fact, be interpreted with commensurable relationships of the musical analogy and proportional means. The discrepancy between the actual villa constructed and that designed in the treatise is also particular (Figure10); regarding today's accepted opinion the designs of the treatise signified the «emblematical and theoretical», i.e. were a sort of ideal model for buildings. All of this, then, justifies their importance rather than their actual measures when analysing Palladio's «architectural principals».

The exceptionality of the «Rotonda» is translated into the intepretation of a villa-temple, deriving from the stateliness of the dome, in addition to some of the most exploited sources of the classical temple. In all of this one perceives, undoubtedly, a suggestion of the Pantheon, consolidated by the testimony of Inigo Jones referring to the eye on the open sky existing in his time in place of the lantern, which is both in the treatise and in the present state of the construction.

There are also references to the Roman shrines from the republican era at Tivoli and Preneste. Palladio, in one of his drawings, re-constructs the shrine at Preneste: as a coronation of the shrine there is the framework of a building comparable to the «Rotonda». The shrine also presents another suggestion which links sacredness and architectural theory relative to, even if non-exclusively, the temple-type. The presence of exedra and cavea areas indicate that theatrical space and religious buildings are parts of a single sacred area. The intrinsic sacredness of the ancient theatre encourages, then, the suggestion of an interpretation of the «Rotonda», which is a comparison to the richness of retort and conceptual overlapping, seen by means of the Vitruvian treatment of the theatre. From this comparison we will discover how a joint description is formed from the various observations posed concerning villa «Almerigo».

The numbers which appear on the plan referred to in the *Quattro Libri* (Figure10) will be our guide with regard to this research. If we assume that the fundamental task of proportioning for a plan characterised by the centrality of a circle had to be, for Palladio, that of co-ordination, harmonisation between the dimensions of the circle and the adjoining rooms on the orthogonal side, one will be struck by the fact that the measurement of 26 feet, which constitutes the largest dimension of the four main rooms corresponding to the four edges of the building, is the length of the side of the equilateral triangle inscribed in the circle of a diameter of 30, which is, in fact, the central circular room.

These four rooms appear, in effect, to correspond to the four equilateral triangles inscribed in the circle, which costitute the basic figure of Vitruvius' (Roman) theatre. The allusion becomes analogy when we consider the orientation of the villa and remember what has previously been stated regarding the symbolic connections between the four astrological trigonals, the winds and the four cardinal points. Each zodiacal trigonal is coupled with a cardinal point in ancient astrology; this coupling is determined by the

dominant wind under the main signs (i.e. in the months governed by those signs) of the trigonals.

With regard to the «Rotonda» we can note — almost as if it were an anomaly— how the cardinal points correspond to the edges and not to the fronts: however, «this orientation is too perfect to be purely casual» (Semenzato 1990). If each of the larger rooms refers to a trigonal it is evident that it is actually the edges where the rooms meet which necessarily point towards the cardinal directions. The justification of the orientation «at the edges» was seen as being correct as a response to the wind direction: «The strongest of the dominant winds in the region comes from the north-east and it was considered better to place it in opposition to an edge rather than a front» (Semenzato 1990). This is a practical justification (moreover, a facade at north-east corresponds, whereas to ancient astrological thinking winds, trigonals and the cardinal points are identical).

The hidden presence —submerged— of the geometries in the ancient theatre becomes even more sensitive if we also consider the other dimensions of the rooms. The main areas, at the angles of the villa, take as co-ordinates the measurements of 26 and 15 feet. Fifteen is, then, the larger dimension of the rooms flanking the corridors, which lead to the circular space, while the smaller dimension is eleven (Figure10). As fifteen is the radius of the central circle, the larger rooms are constituted by proportions which express the harmony of the circle with the triangle inscribed within it; it is impossible not to grasp the precise musical sense of this connection after having examined the Ptolemaic analogies of regular polygons inscribed upon a circle.

As for the dimensions of the smaller rooms (Figure10), 15 and 11 give us the sum of 26. The division of the larger side of the rooms at the angle of these two unequal parts is also very simple, obtainable from the figure of the four triangles inscribed upon the circle. In fact, the intersection of the two corresponding sides pertaining to two consecutive triangles divides the sides in question exactly according to those proportions.

Essentially, the construction of the four triangles inscribed in the circle appear to harmoniously support not only the relationships between the central room and the main rectangular rooms, but also the relationships and the passage, through the smaller rooms, between the four adjacent rooms: one passes from one room to another through smaller rooms which average out the relationships between the larger rooms in the same way in which the intersections of the sides of the triangles average out the relationships of the triangles among themselves.

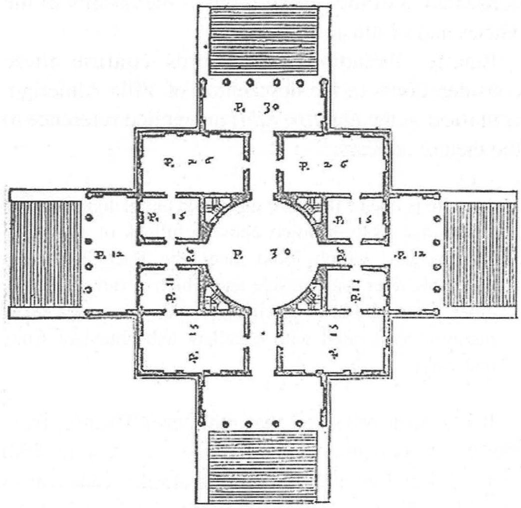

Figure 10
Rotonda's plan from *Quattro Libri*

The numbers 12 and 16 are also introduced into this coherent square: the four trigonals, in fact, subdivide into twelve parts (adjoining the apexes one obtains a dodecagon). So it is also possible to discern an allusion to temporal universality: this aspect is, moreover, obvious (also) when dealing with astrological analogies.

The interpretation emanating from the numbers of the treatise also corresponds with other various aspects of the villa and the description given by Palladio himself. The musical relationships, which the central circle intuits in the adjoining rooms are such that the relationship between 15 and 26 can also be read as in that between the inscribed equilateral triangle and the hexagon (whose side, equal to the edge of the circle is exactly 15): this relationship corresponds, musically, to an octave. This irradiation of harmonious relationships of the circular room to the other rooms is in accordance with the

consecration of the central room as the locality of the Muses and of music.

Finally, Palladio's own words confirm these considerations. In the description of Villa Almerigo, contained in the *Quattro Libri* an explicit reference to the theatre appears:

> The site is one of the most agreeable and delightful to be found, due to its position above a hillock of extremely easy access, which looks over the Bacchiglione, a navigable river, on one side and which is surrounded by other pleasant hills —giving the idea of a *very large theatre*— cultivated with excellent and abundant fruits and vines . . .

It has been observed that «the term Theatre, from the Greek θεαομαι "contemplate", is dear to 15th century literature and refers to the ample visual aspect provided by the hills» (Assunto 1990). Used by Palladio, however, we can assume that it offers more significance than that generally expected from a literary *topos*. The necessity arises, spontaneously, at this point for us to refer to Palladio's familiarity with the Vitruvian treatment of the theatre and, in an even more conclusive manner, with regard to the conceptual merit of this model relative to his «Teatro Olimpico» in Vicenza.

If the musical and astrological analogies of the ancient theatre extend from the building to nature, extending even further across a cosmic dimension, revealing, in this way, its own harmonious character of a «grand theatre», the sentiment contained in the words used by Palladio to describe his work appear to follow a contrary path: construction is concentration, resonance in a restricted space within that natural «grand theatre»; it is the intensification of a force field which embraces and governs both terrestrial nature —the landscape— and mathematical, geometrical and musical nature, which expresses itself on a cosmic scale, it is the «talisman» of the world.

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Between geometry and mechanics: A re-examination of the principles of stereotomy from a statical point of view

Danila Aita

The main objective of this paper is to give a mechanical interpretation of the geometrical principles guiding the art of stereotomy for designing masonry arches. The treatises on the *coupe des pierres* —even those published after the birth of modern structural mechanics— deal with the design of vaulted structures from an essentially geometrical point of view. For instance, the main issue of cutting voussoirs, as concerned the inclination of the joints, was dealt with in geometrical terms without taking any statical consequences into account. With reference to this problem, the *coupe des pierres* develops two geometrical criteria: the first requires that the joints converge at a single point (e.g. Villard de Honnecourt); the second requires that the joints be perpendicular to the intrados of the arch (e.g. Frézier).

In order to determine the degree of stability corresponding to these geometrical criteria, the present paper analyses the problem of stonecutting in statical terms by considering the equilibrium of voussoirs in the absence of friction and cohesion. The works of Coulomb, de Nieuport and Venturoli are examined and the statical formulation of the problem is extended to some stereotomic constructions.

The origins of stereotomy

From the Middle Ages to the 18th century, stereotomy was considered the most important construction technique. By means of geometrical principles, in fact, it allows one to visualize a tridimensional object by means of a bidimensional reproduction and to give an appropriate form to each of the voussoirs making up a vault. In this way, it is possible to construct vaults, domes and squinches and to perform an infinite variety of bold technical operations.

In this context, it is interesting to observe that the design of complex vaulted structures seems to hark back simply to the solution of geometrical problems. In antiquity, the arch was considered as a pre-eminent example of geometrical perfection, containing in itself a principle of statical perfection: the common conviction was that geometry, not statics, could provide the safest proportions for designing arches.

The ancient Egyptians, Greeks and Romans cut stones into large blocks, so that they formed sound constructions and their weight took the place of mortar.

With the passing of time, efforts were made to reduce the dimensions of the stones constructing the structure, so as not to place excessive organisational demands on the building site. Hence the first objective in perfecting techniques for cutting stone is finding stability comparable to that which would be obtained using much bigger stones, while using smaller ones.

A second problem relating to stonecutting is linked to the fact that stone is characterised by a high resistance to compression and a low resistance to traction and to bending. For this reason in ancient temples the maximum distance between the axes of the columns did not exceed 4–5 metres. Hence the

second objective in improvement of hewn stone construction techniques is to solve the problem of getting over bigger inter-axis spaces and coverings. The so-called *encorbellement* method (Fig. 1a) was the first solution, used starting from antiquity. The construction principle is very simple: it consists in using overhanging (i.e., corbelled) stones, with the beds always horizontal.

Though this technique may appear unrefined and primitive, it made it possible to realise some works of inestimable value. One of the most ancient and celebrated was the so-called room of «The Treasury of Atreus», a masterpiece of Mycenean architecture done in the 13th century BC. From the 7th to the 2nd century BC the Etruscans frequently used corbelling to cover some funerary chambers (one thinks, for example, of the tombs at Casale Marittimo and Montagnola) or to make arches (Sakarovitch, 1998).

While *encorbellement* is a technique that came into being for constructing hewn stone structures, the arch and the tunnel vault came into being as brick constructions. They appeared starting from the beginning of the 3rd millennium in regions where there was a shortage of wood, like Mesopotamia and the valley of the Nile, but came to be part of the stonecutting technique only with the introduction of the *voussoir*, a wedge-shaped stone with two oblique faces by means of which it rests on the adjacent voussoirs, laterally transferring the vertical forces due to its own weight and any other loads.

The first examples of arch structures in the Greek-Roman world, whose dating is certain, do not go back to earlier than the end of the 4th century or the beginning of the 3rd. We are referring to the arches that cover the gates of fortifications at Eraclea of Latmos and at Velia, which were ancient Phocian colonies in Central Italy, or the ones under the vaulted rooms of some Macedonian tombs (Langhada, Leucadia) or again the underground chambers of the theatre at Alinda in Caria. In these different examples, as in the Egyptian vaults, the problem posed by the lateral dissipation of the thrusts exerted by the vault or by the arch is solved, in that the vault belongs to a structure that is interred or the arch covers an aperture belonging to a wall. Down to the 2nd century BC all structures with arches or vaults are of this type. This is still the case in the very beautiful vaults of the staircase at the Pergamos *Gymnasium*. It was the Roman builders that, starting from the end of the 2nd century BC, first made the vault a free volume: with them, the vault showed itself openly, came out of the ground, and became a noble construction, no longer confined to subterranean constructions and funerary architecture (Sakarovitch, 1998).

At all events, the cradle of stereotomy was palaeo-Christian Syria. In the middle of the 3rd century AD the *Philippopolis* theatre was built on the Jebel ed-Druz: it contains some rampant arches and a cross vault. Theodoricus' mausoleum is the only Italian monument comparable, for stereotomic virtuosity, to the constructions of palaeo-Christian Syria mentioned —indeed, it is even supposed that the architect originally came from Syria (Adam, 1984, 207).

Hence skilful *architecture clavée* came into being at the confines of the Roman and later Byzantine Empire, an area where, for defence against Persian invasions, the most elaborate fortification systems were built. The encounter in the same region between a long tradition of stone construction, the knowledge of the best Roman architects and engineers and specific demands of military architecture can perhaps explain the perfecting of local craftsmen in the realisation of arch or vault structures (Mango, 1993).

According to a hypothesis based on nineteenth-century studies by Viollet-le-Duc (1854–1868) and Choisy (1873; 1883), stonecutting methods appear to have been brought from the East to the West by crusaders. The development of stereotomy in the South of France in the 12th and 13th centuries is one argument in favour of this thesis (Sakarovitch, 1998).

The first problem that faced medieval builders in the realisation of vaults was how to cut the voussoirs constituting a structure. They seem to have answered this question from an essentially geometrical point of view, without taking statical or structural considerations into account. Indeed, stereotomy treatises illustrate the rules according to which voussoirs are to be cut in order to solve the different *geometrical* problems that may arise.

The various methods with which stones can be cut can be grouped into two big families: on one hand, archaic methods, and on the other hand cutting *par équarrissement* and *par panneaux*. Archaic methods are those which require no preparatory trace. There are essentially three: cutting *par ravalement*, *à la demande* and *à la perche* (Sakarovitch, 1998).

Cutting *par ravalement* (Fig. 1b) consists in cutting the stones when they are in place in the vault.

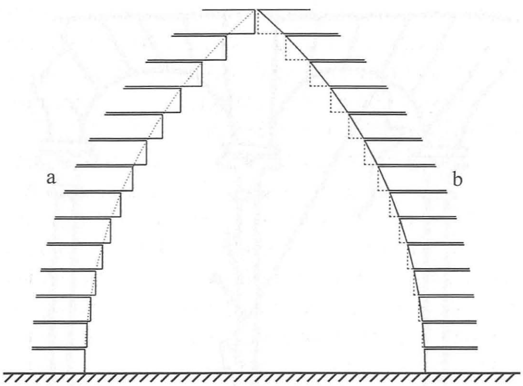

Figure 1
The *encorbellement* method (a) and the method of cutting *par ravalement* (b)

Before they are put in their definitive position, they are roughly hewn, and only when they are in their definitive position are they given their exact shape. For example, in the room of «The Treasury of Atreus», where the *encorbellement* technique was used, the intrados of the vault was cut after the stones were put in place, the excess stone which formed a sort of upside-down staircase being removed, with a cutting method *par ravalement*. Closer to sculpture than to stereotomy, this technique presents two disadvantages. On the one hand, it makes it necessary to put in bigger stones than are necessary and to cut them afterwards in difficult working conditions. On the other hand, the *ravalement* removes the mortar and hence it can only be used in constructions *à joints vifs* (Sakarovitch, 1998; Choisy, 1899).

In cutting *à la demande*, each stone is hewn for subsequent retouching, in relation to the *claveaux* already put in place on which it is to rest. This type of technique, used for example in Romanesque architecture, is very slow. The advantage is a great versatility of use, with relatively little material and work, since it is possible to choose for each case the rough stone that best approximates to the *claveau* to be made (Sakarovitch, 1998; Chappuis, 1962).

Probably in order to accelerate the speed of construction on sites, cutting techniques were perfected and better exploited the potentialities of geometry.

Cutting *par équarrissement*, also known as *derobement*, consists in cutting the stone without the help of *panneaux*, using the heights and depths delimiting the voussoir to be made.

With the method *par panneaux*, instead, the

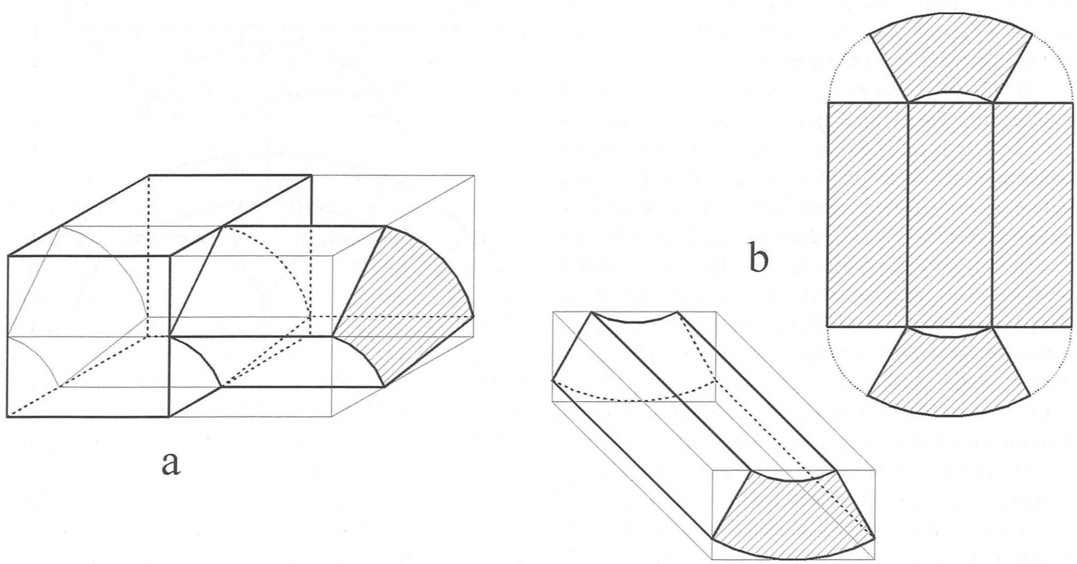

Figure 2
Cutting *par équarrissement* (a) and *par panneaux* (b)

volume of each voussoir is determined starting from the surface of each of its faces. Efforts are made to inscribe a voussoir in the smallest possible parallelepiped rectangle. In order to do this, the parallelepiped can be rotated at a certain angle with respect to the vertical. All references to it having been lost, it is necessary to use *panneaux*, i.e. models reproducing the shape of the faces of the voussoir with the true dimensions.

CUTTING VOUSSOIRS: A GEOMETRICAL PROBLEM OR A STATIC ONE?

In order to highlight the peculiarities of stereotomy, which lies somewhere between geometry and structural mechanics, it seemed particularly useful to analyse some of the main treatises on *coupe des pierres*, dwelling in particular on one problem: the determination of the inclination of the joints when the arch intrados and extrados have been assigned.

Regarding the *tailleurs de pierre*, I have identified two main «schools of thought».

A first theory maintains that the straight lines representing the direction of the joints must converge at a point, whatever the arch intrados and extrados curves are like. This theory is found, for example, in Villard de Honnecourt (13th century) and in Milliet Dechales (1674). It is based on the executive simplicity of the use of a rope to mark out the traces of the joints, but takes into account neither constructive nor statical factors (only in the case of the platband, as we shall see, does the theory correspond to a correct *statical* solution to the problem). Perhaps it was precisely because of the lack of consideration for construction problems that this theory did not enjoy great favour. The fact is that it contemplates the possibility of realising both acute and obtuse angles in cutting the stone, and this certainly constitutes an element of executive difficulty and construction weakness.

In a sketch by Villard (Fig. 3), we find an explanation of how to trace out the wedges of a pair of arches with a suspended intermediate capital, using a rope to mark out the traces. In this case —examined also by Milliet Dechales (Fig. 4)— arch-capital-arch is assimilated to a single vaulted structure.

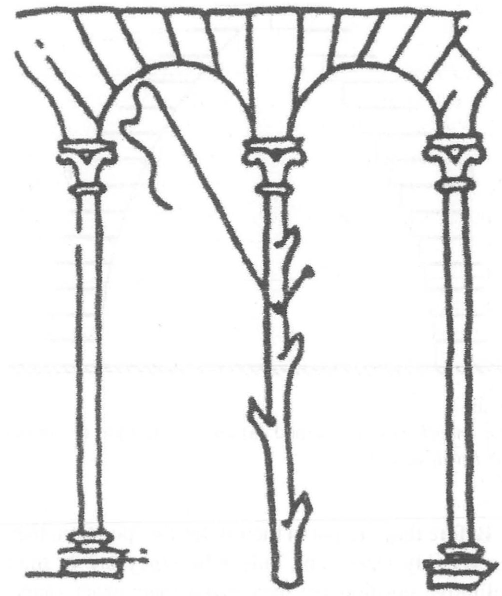

Figure 3
Villard de Honnecourt's *Carnet* (13th century): tracing out the voussoirs of a pair of arches with a suspended intermediate capital

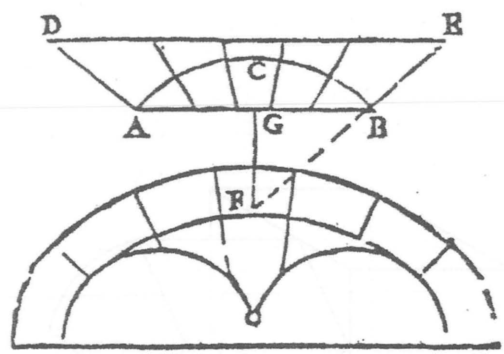

Figure 4
Milliet Dechales (1674): *De arcu in alias figuras degenerante*

A second theory, instead, maintains the perpendicularity of the joints to the intrados line (1). This theory is present, for example, in Frézier (1737–1739). It is excellent from the construction viewpoint, since the right angle is the easiest to execute and the most uniformly resistant.

In Frézier's treatise, stereotomy is viewed as a set of prevalently geometrical rules. For Frézier the expression *coupe des pierres* does not so much mean «. . . l'ouvrage de l'artisan qui taille la pierre», as «la science du mathematicien, qui le conduit dans le dessein qu'il a de former une voûte, ou un corps d'une certaine figure par l'assemblage de plusieurs petites parties». In problem 26 of Ch. IV of Book II, the subject of the tracing of the *joints de tête* is dealt with. In the case of round arches, it coincides with that of the construction of a perpendicular to an arc of a circle, passing through an assigned point. Stonemasons call this operation *le trait quarrée sur la ligne courbe, et au but de la ligne courbe.*

Frézier notes that stonemasons also apply this method for arches formed by portions of ellipses or of other curves.

In problem 27 of Ch. IV of Book II, defined by Frézier as *manière de tracer les joints de tête des ceintres fait d'arcs de sections coniques*, there is studied the way to trace a perpendicular to an arc of a conical section passing through a point lying on the same section.

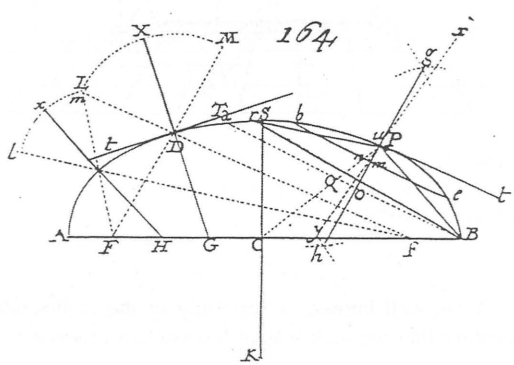

Figure 5
Frézier (1737–1739): the hypothesis of orthogonality of the joints to the intrados line

But what are the reasons for a similar choice?

The first reason is related to stability and statical equilibrium: the *têtes* of the voussoirs become «de sorte que la pierre ne peur passer par l'ouverture inférieure de l'intervale de deux voussoirs, qui est plus étroit à la doele qu'à l'extrados; ainsi étant pressée par sa pesanteur contre les voussoirs collateraux, qui se servent mutuellement d'appui les uns aux autres, elle est soutenue en l'air par la résistance des derniers appuis, qui sont les piedroits, lesquels doivent avoir assez de force pour contrebalancer l'effort que ce voussoirs ou especes de coin font pour les écarter».

The second reason is related to symmetry and the need not to create dishomogeneity in the distribution of forces, «afin de conserver toûjours une inclinaison uniforme des joins de tête sur la courbe du ceintre; car quand même les parties de l'arc exterieur et de l'interieur ne seroient pas proportionnelles, la voûte n'en subsisteroit pas moins, pourvû que celles de l'interieur soient toujours plus petites que celles de l'exterieur, il n'en résulteroit d'inconvenient que de la difformité, et une inégale impulsion des voussoirs contre leurs collateraux».

The third reason has to do with motivations of a constructive character and observations relating to material resistance: «parce que les plans que passent par les joints de tête, qu'on appelle les lits, étant perpendiculaires à la tangente de l'arc au point de sa division, font avec la doele de part et d'autre le plus grand angle qu'ils puissent faire, qui est le droit, ou infinitement peu different du droit; car si on le faisoit obtus d'un côté, il rendroit l'autre aigu. Or il importe que les résistances des arêtes, c'est à dire, des angles des Pierres, soient égales pour porter également la charge, car il est clair que la plus forte feroit casser la plus foible, comme l'expérience le fait voir aux platebandes, où lon est forcé d'en agir autrement».

Hence, for Frézier, perpendicularity of joints to the intrados is not a prejudice accepted *a priori* but a guarantee of constructive resistance, so much so that, paradoxically, the fact that in a platband the straight lines of the joints have to converge at a point forming acute angles is seen as involving a risk of greater weakness of the *angles des pierres*, and not as the statically correct solution later demonstrated by Coulomb (1776) in the case of the absence of friction or cohesion between voussoirs. Frézier says in Ch. IV of Book IV of Tome III: «On peut tracer l'épure de cette espece de voûte de plusieurs manières, qui reviennent toutes à la même fin, dans lequelles il y a plus de disposition de goût que de Géometrie, & l'on peut dire que la solution de ce problême est assez arbitraire pour la détermination de l'inclinaison des

joints de lits». Actually the solutions presented by Frézier correspond to the statically correct one proposed by Coulomb (Fig. 6).

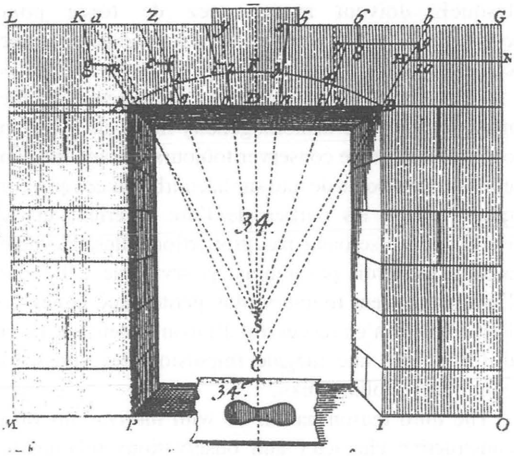

Figure 6
Tracing of the inclination of the joints in a platband for Frézier (1737–1739)

The geometrical character of stereotomy, at least as it was conceived down to the start of the eighteenth century, culminated in the treatise by Desargues (1640), who applied his «universal methods» to the technique of stonecutting, endeavouring to solve the particular problems of stereotomy with a single rule. Unlike what happened in all other treatises on stereotomy, which until the 19th century were presented as more or less complete collections of cases, Desargues studies a single architectonic object, the *descente biaise dans un mur en talus* (Fig. 7). The term *descente* indicates a type of cylindrical vault whose axis is not horizontal; the term *biaise* implies that the angle between the axis of the vault and the wall *en talus* (not vertical) is generic.

After defining the technical terms, Desargues defines the planes and straight lines that will be required for reference: the *plan de face*, which is the plane of the wall; the *essieu*, which is the axis of the tunnel vault and gives the direction to the generatrices; the *plan droit à l'essieu*, which is the plane perpendicular to the *essieu*, which bears the *section droite* of the vault.

After setting up these preliminary hypotheses, Desargues seeks to solve the geometrical problem of obtaining the true dimensions of the faces (or of the angles) required for cutting the stone.

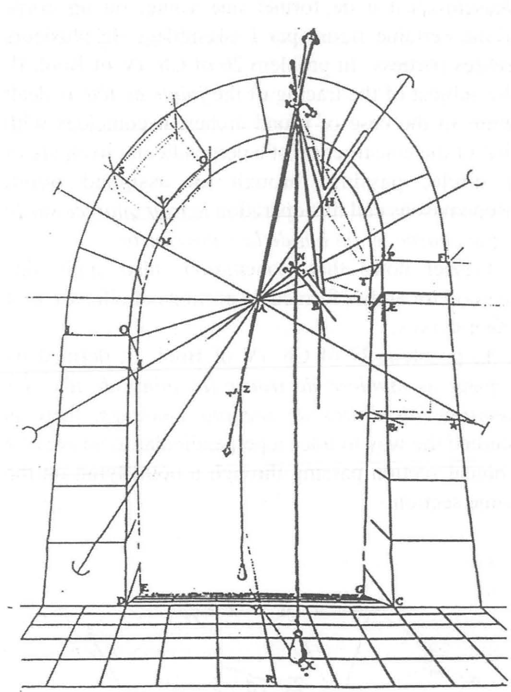

Figure 7
The *descente biaise dans un mur en talus* studied by Desargues (1690)

As is well known, it was only in the eighteenth century that the arch was at last studied in a statical key.

Philippe De La Hire, a versatile and illustrious French scholar known for his *Traité de Mecanique*, is commonly remembered as the first author to have dealt with the theme of arches and vaults from a statical point of view. Indeed, later scientists in the 18th and 19th centuries referred to him, considering his theories as first more or less successful attempts to use mechanics to account for construction rules, which until that time had been entrusted to practice and intuition.

Philippe De La Hire was a disciple of Desargues, and dealt with mechanics, astronomy, mathematics

and engineering. He was an outstanding member of the Académie Royale des Sciences; taught mathematics at the Collège de France and also gave lectures at the Académie d'Architecture. There are no printed versions of these lectures, but two manuscripts: *Architecture Civile* e *Traité de la coupe des pierres*; the latter was a subject he taught for over twenty years.

The *Traité de la coupe des pierres* (late 17th century) has not been published; however, Frézier, in his *Traité de Stéréotomie*, takes up some topics from it. In De La Hire's manuscript we find the most common arguments relating to stonecutting, but the geometrical constructions are very complex. Of major interest is the start of the treatise, where De La Hire affirms that «Les ouvriers appellent la science du trait dans la coupe des pierres, celle qui enseigne à tailler et à former séparément plusieurs pierres, en telle sorte qu'étant jointes toutes ensemble dans l'ordre qui leur est convenable, elles ne composent qu'un massif qu'on peut considérer comme une seule pierre». In this passage (for the first time in a treatise on stereotomy) it is stated that a necessary condition for the stability of a vaulted structure is the absence of kinematic motions between the parts, i.e. equilibrium between the parts.

As regards the inclination of the *joints de tête*, from some drawings present in the *Traité de la coupe des pierres* it can be observed that it must be perpendicular to the tangent to the intrados curve in the point of division of the joint. The hypothesis of orthogonality of the joints to the intrados was also to persist in the two works on mechanics by De La Hire, i.e. his *Traité de Mecanique* (1695) and his subsequent memoir of 1712 entitled *Sur la construction des voûtes dans les edifices* (1731). In these works reference is made to two fundamental problems: one relating to the figure of the arch and the other concerning the sizing of the piers. In the *Traité* there is an intuition, though a confused one, of the pathway that was soon to lead to the solution of the first problem; the 1712 memoir offers the first imperfect but promising solution which through successive passages was to lead in future to collapse calculation.

Perhaps precisely because statical approaches to the arch were inaugurated by a scholar coming from the world of stereotomy, the orthogonality of the joints to the intrados appears like an implicit hypothesis in the considerations of almost all authors that deal with vaulted joints, from that time down to Coulomb, such as Charles Bossut, Claude Antoine Couplet, Giordano Riccati, Mariano Fontana and Anton Maria Lorgna.

In the panorama of historical treatises on arches and vaulted structures, it is interesting to observe that the problem of the inclination of the joints in an arch is only studied from a statical point of view by a few authors, such as Coulomb, De Nieuport e Venturoli.

In his *Essai* (1776), Coulomb sets out to solve the problem of determining the direction of the joints in a vaulted structure whose intrados and extrados curves have been assigned, so that the structure will be in equilibrium in the absence of friction and cohesion between the joints.

Let P e Q(↲) be the components, horizontal and vertical respectively, of the resultant of the forces acting on the part *aGMq* of the vault (Fig. 8).

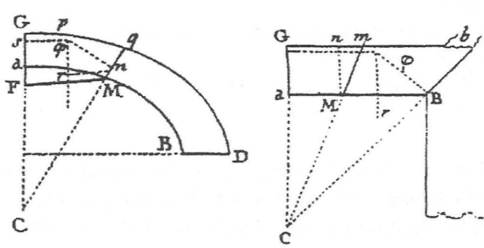

Figure 8
The problem of the inclination of the joints according to Coulomb (1776)

There are two conditions to respect for the vault to be in equilibrium in the case of the absence of friction and cohesion between the joints:

— the resultant must be perpendicular to the joint *Mq*, whose direction forms an angle • with the vertical; i.e. it must be:

$$Q(\psi) = P \tan \psi \qquad (1)$$

— the resultant must always pass between the points M and q.

As anticipated, Coulomb shows that in a platband the straight lines of the joints have to converge at a point.

In his *Elementi di Meccanica* (1806), Venturoli goes back to Coulomb's treatment of the equilibrium of arches in the absence of friction and cohesion between the joints, with the intention of proposing a rereading of the problem in differential terms.

Venturoli considers an arch *E'aE*, symmetrical with respect to the vertical axis *AB*, made up of infinite voussoirs weighing *MmnN* «contiguous, but not connected to one another» and resting on the motionless «pulvinars» without friction and cohesion *Ee*, *E'e'* (Fig. 9).

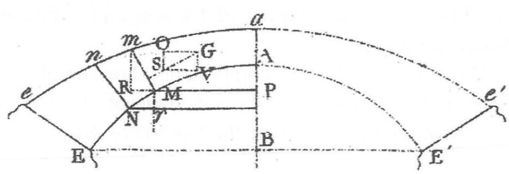

Figure 9
The problem of the inclination of the joints according to Venturoli (1806)

Let one project orthogonally the intrados curve *AME* on the vertical axis *AB*. The generic point *M* will be identified by the coordinates $AP = x$ e $PM = z$; let one denote with $h(\bullet)$ the length of the generic bed *Mm*, to which there corresponds the angle • and the coordinates (z,x).

Calculating by means of analytical trigonometry the area of the infinitesimal quadrilateral *MmnN*, comprised between the joint *Mm*, identified by •, and the joint *Nn*, identified by • $+d$•, we obtain:

$$\text{Area } MmnN = \frac{1}{2}\, h(\psi)^2 d\psi + h(\psi)[dx \sin \psi + dz \cos \psi] \tag{2}$$

The area of *MmnN*, calculated in (2), is proportional to the weight dQ of the infinitesimal voussoir. For (1), we will have:

$$dQ = \frac{P d\psi}{\cos^2 \psi} \tag{3}$$

Hence we obtain the equation:

$$\frac{1}{2}\, h(\psi)^2 d\psi + h(\psi)[dx \sin \psi + dz \cos \psi] = \frac{P d\psi}{\cos^2 \psi} \tag{4}$$

by means of which, knowing the intrados curve and the law of the inclination of the joints, it is possible to calculate the length $h(\bullet)$ of each joint *Mm* and hence the thickness of the arch; or, vice versa, if $h(\bullet)$ is assigned, it is possible to find the direction of the joints, in order to satisfy the first equilibrium condition.

Another scholar that considered the influence of voussoir cutting on the equilibrium of a masonry arch was de Nieuport (1781). Starting from De La Hire's theorem, he considered the fact that, in general, in an arch there are three fundamental curves: the intrados, the extrados and the curve formed by the intersection points of the straight lines of the joints. De Nieuport studied not only cases in which the joints are orthogonal to the intrados or converge at a point, but also more general cases. It is necessary, then, to make reference to the curvature radius and consider the voussoirs as infinitely small but having thickness. The three fundamental curves are connected to one another by the equilibrium relations. Knowing two of them through the equilibrium conditions, one determines the third curve (Radelet de Grave, 1995). The memoir continues with the elaboration of complex analytic developments, backed up by graphic results (Fig. 10).

It is thus possible to determine the law governing the inclination of the joints in vaulted structures having the intrados and extrados assigned, so as to

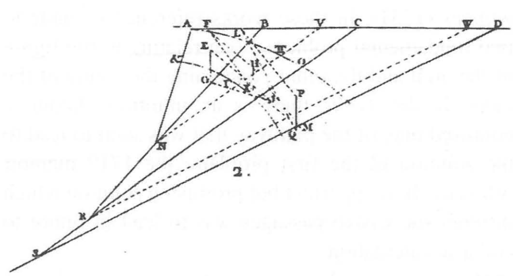

Figure 10
De Nieuport (1781): relation between the intrados curve, the extrados curve and the curve formed by the intersection points of the straight lines of the joints

ensure the equilibrium in sliding, even in the absence of friction and cohesion between the voussoirs.

Hereafter, without illustrating the mathematics of the problem, I give here some graphic results for some structural typologies present in stereotomy treatises. The geometrical constructions proposed by the *tailleurs de pierre* presuppose that the straight lines representing the inclination of the joints will converge at a single point, or will be orthogonal to the intrados. However, the equilibrium solution in the absence of friction and cohesion between the joints does not coincide with the stereotomic solution (Aita, 2001). By way of example, in the case of a circular arch without friction or cohesion between the joints, equilibrium in sliding is ensured if they are inclined as in Fig. 14: hence they will not prove perpendicular to the intrados (a hypothesis always implicitly considered both in the stereotomy treatises and in the statical ones before Coulomb, Figure 13).

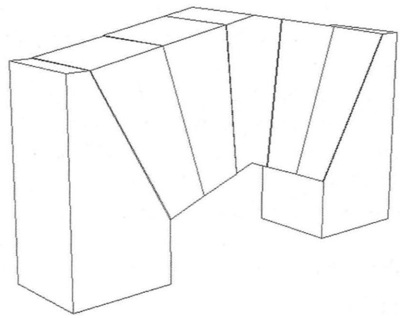

Figure 12
Inclination of the joints in a platband with extrados and intrados *en chape*: equilibrium solution in absence of friction and cohesion

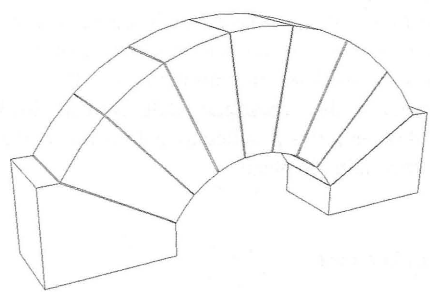

Figure 13
Inclination of the joints in a circular arch: the hypothesis always implicitly considered both in the stereotomy treatises and in the statical ones before Coulomb

CONCLUSIONS

In order better to understand the delicate relationship between stereotomy and mechanics, it is perhaps useful, at the end, to observe that the arch model adopted by Coulomb, de Nieuport and Venturoli for the inclination of the joints from a statical point of view is that of a system of rigid heavy blocks, perfectly smoothed and devoid of friction, analogous to the one first proposed by De La Hire. In effect, the presence of friction and cohesion ensures the stability of vaults made in accordance with the principles of stereotomy. At all events, it is interesting to observe that statical modelling —*à la De La Hire*— did not influence *la théorie et la pratique de la coupe des pierres*, which instead developed on the basis of geometrical principles and empirical rules that sedimented in the course of time, permitting the construction of architectures of inestimable value and, paradoxically, of great structural interest.

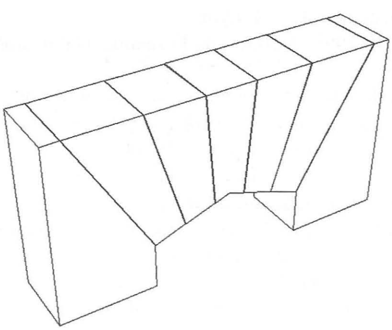

Figure 11
Inclination of the joints in a platband with a horizontal extrados and an intrados *en chape*: equilibrium solution in absence of friction and cohesion

AKNOWLEDGEMENTS

A special thank to Geom. Gabriele Mazzei for his help in preparing the graphic material of this paper and for his practical suggestions.

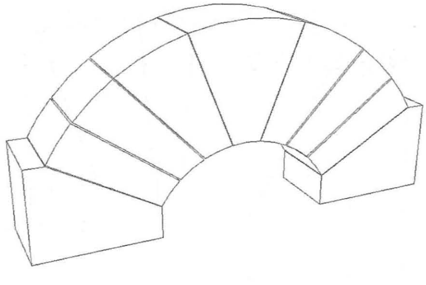

Figure 14
Inclination of the joints in a circular arch: equilibrium solution in absence of friction and cohesion

Notes

The term *intrados line* refers to the curve determined by the intersection of the intrados surface of the vault and a plane used to draw up the *épure*. It is usually orthogonal to the axis of the vault, but can also be a vertical plane if this is suited to making the *trait* easy, or another suitable plane.

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Some explicit solutions for flat and depressed masonry arches

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R. Barsotti
S. Bennati

Concerning the analysis of the mechanical behaviour of masonry arches, two main lines of theoretical investigation can be distinguished: the first regards arches as a system of rigid voussoirs subject to friction and unilateral constraints and attempts to evaluate the system's distance from collapse conditions. The second, following instead a continuum mechanics approach and an *«elastic»* or *«pseudo-elastic»* logic, aims to determine the evolving stress and strain fields in the arch. Within this framework, the greatest difficulty has been modelling the behaviour of very complex materials, in particular, masonry. In fact, masonry, a heterogeneous, anisotropic material made up of blocks and some binding matrix, is characterised by sufficiently high resistance to compression, but low resistance to tension. The comprehensive constitutive laws able to provide detailed mathematical descriptions of the behaviour of such materials often prove to be extremely complex.

Following the work of Signorini (1925b), who performed the first accurate studies of elastic materials incapable of withstanding significant tensile stresses, some authors have proposed a non-linear elastic constitutive relation (Di Pasquale 1982, Del Piero 1989, Angelillo 1993). According to this view, masonry arches can be studied as one-dimensional, non-linear curved elastic beams, so that describing their mechanical behaviour becomes a matter of solving non-linear ordinary differential equations. In fact, if the structure is isostatic, a simple integration leads to determination of explicit expressions for the displacements and rotation of any given point on the line of axis. In this case, the line of thrust and the possible arch regions of non-linear behaviour are known. If the structure is statically indeterminate, however, the displacements and rotations must be expressed as functions of the unknown redundant reactions, and then restraining conditions imposed, which leads to a non-linear algebraic system (Bennati and Barsotti 1999; 2001).

In the case of «simple» redundant structures, such as flat arches or depressed arches with clamped ends, the solution can be found explicitly: it is in fact possible to arrive at explicit expressions for both the displacements and the rotations. Moreover, the boundaries of the regions which behave non-linearly can be determined analytically. For such vaulted masonry structures, the evolution of the stress and strain fields under increasing loads can be easily studied. Collapse is reached when a kinematic mechanism emerges and the residual stiffness of the arch vanishes.

A brief historical look

As early as the 18th century the mechanical behaviour of masonry arch structures attracted a great deal of interest on the part of researchers, interest which

continues unabated today. A first line of reasoning, whose origins date back to some eighteenth-century research (La Hire 1712; La Hire 1730; Coulomb 1776), conceives of the arch as a system of rigid blocks and focuses on the mechanisms of collapse and determination of the ultimate load. A second approach instead seeks to investigate the evolution of the stress and strain fields with increasing applied loads. The first researcher to deal with arches as deformable continua was the renowned French engineer Navier. In his 1826 *Résumé des Leçons*, Navier sets forth a theory of arches based on the limit analysis of a system of rigid blocks, after which he adds the following observation:

> Dans la réalité les voussoirs n'étant parfaitement durs, on ne peut admettre que les pressions s'exercent ainsi contre des arêtes. Cela n'empêche pas que l'on ne puisse calculer, avec une exactitude suffisante, l'équilibre des voûtes d'après les règles énoncées précédemment ; mais il paraît nécessaire d'avoir égard à l'élasticité de la matière des voussoirs. Cette question serait un cas particulier d'une question plus générale, qui consiste à déterminer les effets qui se produisent dans un corps élastique de figure quelconque, soumis à l'action de diverses forces (Navier 1826, 164).

Navier's idea of regarding masonry arches as elastic structures arose in an already fertile scientific context. One need only consider his text, *Mémoire sur les lois de l'équilibre et du mouvement des solides élastiques* (Navier 1827), which offered an important contribution to defining the bases of the modern theory of elasticity. At the same time, Cauchy was hard at work studying elastic bodies, resulting in his volumes, *Exercices de Mathématiques* (Cauchy 1826), and Poisson on establishing his mathematical theory of elasticity, proposed in two fundamental papers (Poisson 1812; Poisson 1829), which were shortly to be followed by the ground-breaking works of Lamé and Clapeyron (1833).

After Navier, little by little the interpretative model of masonry arches as hyperstatic elastic systems came to be established. Studies based on the principles of minima, whose approach was primarily abstract and theoretical, alternated with work grounded on experimental considerations, which offered rapid methods for overcoming the static indeterminacy inherent in such problems. In the early 19th century, F. J. Gerstner (1831, 405) introduced for the first time the concept of the line of thrust, which was widely utilised by subsequent authors. It is a well-known fact that for an arch in equilibrium under assigned loads, one can trace an infinite number of admissible lines of thrust, of which only one is the «true» one. However, in his attempt to determine this true line of thrust, Gerstner introduced further hypotheses, which turned out to be completely arbitrary.

Of the many authors that took up Gerstner's ideas and developed them, those most worthy of note are Moseley (1843) and Scheffler (1857), whose solutions were based on the principle of least resistance, and Méry, who in his celebrated work (Méry 1840) put forth a simple method for determining the value of the horizontal thrust at the keystone under the hypothesis that tensile stresses are absent in every section of the arch. We should also recall the contribution of Drouets (1865), who proposed a «principle of minimum action», according to which the stress state occurring within an arch is that corresponding to the lowest value of the maximum compressive stresses, as well as those of Villarceau (1854), Carvallo (1853), Denfert Rochereau (1859), Saint-Guilhem (1859) and Dupuit (1870), all of whom added to our understanding of the behaviour of masonry arches. However, in this framework, one of the most important contributions came from Durand-Claye, who introduced the method of «stability areas» (Durand-Claye 1867, 66), by which it is possible to determine the set of admissible lines of thrust via a graphical reconstruction, while accounting for both the structure's overall stability, and the limited resistance of the masonry material.

A change in perspective ensued from the lines of reasoning followed by Crotti, who maintained that the position which the thrust lines will assume within the arch can only be identified by considering the material's elastic properties, the mortar's degree of stiffness, the varying thickness of the vaults and joints, in short, all those factors that contribute to defining the «internal constitution of the vaults» (Sinopoli and Foce 2001). In fact, Crotti writes: «ogni altro metodo è inevitabilmente fallace in quanto la vera teoria della resistenza delle volte può essere fondata solo sulla teoria dell'elasticità opportunamente modificata e semplificata» [all other methods are inevitably unsound, in that the true theory of the resistance of vaults can be based only

upon the theory of elasticity, suitably modified and simplified] (Crotti 1875). He was moreover the first to introduce clear equations for kinematic compatibility and constitutive equations, thereby succeeding in furnishing a rational solution to the associated problem of static indeterminacy. It is not surprising that Crotti's choice of methodology was taken up again toward the end of the 19th century by Europe's most illustrious structural engineers, including Italy's Alberto Castigliano (1879).

A well-known fact about masonry is that although it presents excellent resistance to compressive forces (of the order of 10 MPa if made out of bricks, while stone offers much more), it has very limited tensile resistance (only a fraction of a MPa). Its mechanical behaviour, radically different under conditions of tension and compression, renders the results obtained via linear elastic analysis highly questionable. In chapter 6 of his treatise, Castigliano proposes searching for an approximate solution by using an iterative procedure, during which process only that part of the structure that was under compression in the preceding step is considered to be reactive. In the second part of the treatise, Castigliano reports the results of his analysis of two masonry arch bridges conducted on the basis of the foregoing considerations: the first is a brick bridge built over the Oglio river, while the second bridge, designed by the engineer Mosca, was built of freestone over the Dora river in Turin. Revelant to this context, Perrodil (1872) proposed applying elasticity theory to the study of masonry vaults after evaluating the results of tests conducted on an arch of 38–metres' span. Some years later, Perrodil would utilise the equations of linear elasticity theory to analyse constant-thickness masonry vaults (Perrodil 1876)

It is however to Signorini that the credit goes for direct-method studies of the mechanical behaviour of elastic materials unable to bear tensile stresses. After addressing various issues linked to the case of combined compression and bending of reinforced concrete, in 1925 he presented two articles to the National Academy of the *«Lincei»*. In the first, he proves a theorem for the existence and uniqueness of the solution for equilibrium problems in the presence of low tensile-strength materials (Signorini 1925a). In it, Signorini examines the case of combined compression and bending of a straight cylinder made of a homogeneous material with only slight, uncertain resistance to tension and characterized by non-linear elastic behaviour. In the second of the aforementioned articles, he tackles the same problem assuming a piecewise-linear constitutive equation for any longitudinal fibre of the cylinder.

In the 1970's, Signorini's ideas began to be taken up again by other researchers, amongst which Giovanni and Manfredi Romano, who in their studies view masonry arches as one-dimensional elements, exhibiting non-linear elastic behaviour. The authors moreover propose a numerical procedure for arriving at the solution, whose convergence they then demonstrate (Romano 1979).

A NON-LINEAR ELASTIC MODEL FOR MASONRY ARCHES

Following Signorini's idea, it seems reasonable to assume non-linear elastic behaviour for any given longitudinal arch fibre. Here, for the sake of simplicity, we have chosen a piecewise-linear constitutive relation, as shown in Figure 1.

The simple elastic stress-strain constitutive relation used here (Fig. 1) is a piecewise linear function which reduces to the basic linear case for stresses comprised within the threshold values σ_c and σ_t, denoting the

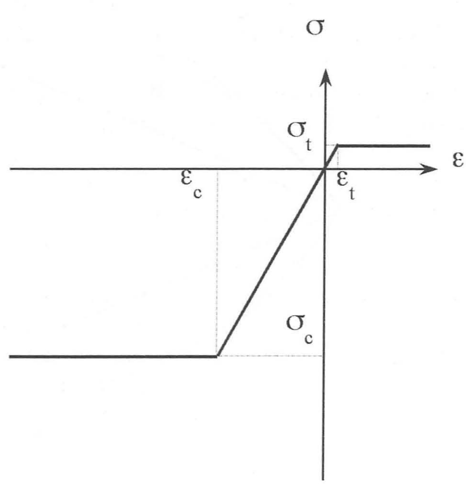

Figure 1
The σ–ε relation

material's resistance to compression and tension, respectively. Outside this range, strain grows at constant stress. Consequently, denoting by E the material's Young's modulus, we have

$$\varepsilon = \begin{cases} \sigma_c & \varepsilon \le \varepsilon_c \\ E\varepsilon & \varepsilon_c < \varepsilon < \varepsilon_t \\ \sigma_t & \varepsilon \ge \varepsilon_t \end{cases} \tag{1}$$

where $\sigma_t = E\varepsilon_t$ and $\sigma_c = E\varepsilon_c$. This simple non-linear equation allows accounting for masonry's weak tensile strength and bounded compressive strength. One drawback is that the material is unrealistically assumed to be able to transmit low tensile stresses, even in presence of high strains.

Constitutive equation (1) was adopted by two of the authors in a previous work (Bennati and Barsotti, 2001), and is an improved version of that used in two other works by the same authors (Bennati and Barsotti 1999; 2002), in which masonry's compressive strength was assumed to be unbounded.

In the following, as usually done in the theory of the bending of beams, the cross-section of the arch, of height h and unit width, is assumed to remain plane and normal to the longitudinal fibres after bending. Moreover, the normal stresses σ_z will depend on the corresponding strains ε_z according to the same relation holding under a monoaxial state of stress. Under the foregoing hypotheses, the strain is linear over any given cross-section.

So, starting with relation (1), we can build the set of corresponding non-linear constitutive equations holding at the cross-sectional level between the kinematics parameters, axial strain ε_0 and curvature χ, and the resultant axial force N and bending moment M.

As the stress-strain relation (1) maps all the strains over a limited range of stress values, it is to be expected that, at any given cross-section, admissible values of N and M will belong to a bounded region of the (N, M) plane. Such region, i.e., the elastic domain, has been plotted in Figure 2.

The three linear parts which make up the stress-strain relation naturally induce a partitioning of the elastic domain in the (N, M) plane into six regions: when the point of co-ordinates (N, M), representing the set of the internal forces at a given section, belongs to region E, then both the strain and the stress are linear. If the point falls instead into regions B (or D), then the stress distribution is non-linear over the compression (or tension) side. When the point belongs to region C, the stress distribution is non-linear over both the compression and tension sides.

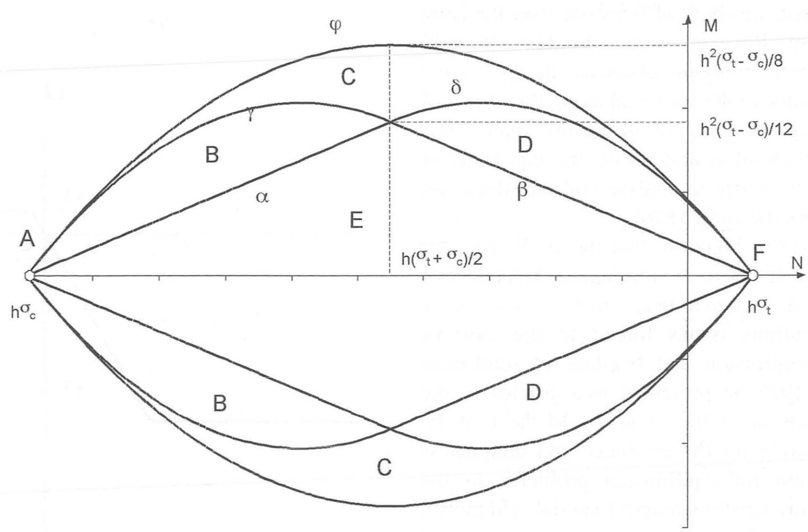

Figure 2
The elastic domain

Finally, when the point coincides with point A (or point F), then the stress corresponds to the limit case where it is constant throughout the cross section and equal to σ_c (or σ_t). As already observed, points situated outside the external boundary of region C cannot be reached. The expressions for the curves separating the different regions of the elastic domain, for $M > 0$, are:

(curve α)

$$6M + Nh - h^2\sigma_c = 0 \tag{2.1}$$

(curve β)

$$6M + Nh - h^2\sigma_t = 0 \tag{2.2}$$

(curve γ)

$$(N - h\sigma_c)(h\sigma_c + 3h\sigma_t - 4N) + 6M(\sigma_c - \sigma_t) = 0 \tag{2.3}$$

(curve δ)

$$(N - h\sigma_t)(4N - 3h\sigma_c - h\sigma_t) - 6M(\sigma_c - \sigma_t) = 0 \tag{2.4}$$

(curve φ)

$$(N - h\sigma_c)(N - h\sigma_t) - 2M(\sigma_c - \sigma_t) = 0 \tag{2.5}$$

In each of the six regions, the strain of the barycentric fibre ε_0 and curvature χ are non-linear functions of the axial force and bending moment. After some algebraic manipulations we obtain, in the case of $M > 0$, the following constitutive relations between the cross-sectional internal forces N and M, and the kinematics parameters ε_0 and χ:

(point A)
$$\begin{cases} N = h\sigma_c \\ M = 0 \end{cases} \tag{3.1}$$

(region B)
$$\begin{cases} N = h\sigma_c\,[1 - E(2\varepsilon_c - 2\varepsilon_0 + \chi h)^2/8\chi h\sigma_c] \\ M = E(2\varepsilon_c - 2\varepsilon_0 + \chi h)^2(\varepsilon_c - \varepsilon_0 - \chi h)/24\chi^2 \end{cases} \tag{3.2}$$

(region C)
$$\begin{cases} N = -h(\sigma_t + \sigma_c)/2 + (\sigma_t - \sigma_c)(2\varepsilon_0 - \varepsilon_c - \varepsilon_t)/2\chi \\ M = h^2(\sigma_t - \sigma_c)/8 - (\sigma_t - \sigma_c)(\varepsilon_c^2 + \varepsilon_c\varepsilon_t + \varepsilon_t^2 - 3(\varepsilon_c + \varepsilon_t)\varepsilon_0 + 3\varepsilon_0^2)/6\chi^2 \end{cases} \tag{3.3}$$

(region D)
$$\begin{cases} N = h\sigma_t\,[1 + E(2\varepsilon_t - 2\varepsilon_0 + \chi h)^2/8\chi h\sigma_t] \\ M = E(2\varepsilon_t - 2\varepsilon_0 - \chi h)^2(\varepsilon_0 - \varepsilon_t - \chi h)/24\chi^2 \end{cases} \tag{3.4}$$

(region E)
$$\begin{cases} N = Eh\varepsilon_0 \\ M = -E\chi h^3/12 \end{cases} \tag{3.5}$$

(point F)
$$\begin{cases} N = h\sigma_t \\ M = 0 \end{cases} \tag{3.6}$$

Analogous relations, omitted here for the sake of brevity, hold in the case of $M < 0$.

By using the one-dimensional non-linear elastic model described in the foregoing, the elastic equilibrium problem of the arch can be written in terms of simple ordinary differential equations.

To this aim, let us indicate by s the curvilinear abscissa along the line of the axis, and let u and v stand for the displacements of points along the axis line in the tangential and radial directions, respectively. Simple calculations, omitted here for the sake of brevity, show that under the foregoing hypotheses, and in the case the arch is circular with radius R, the radial displacement $v(\theta)$ is a solution to the differential equation

$$v'' + v = -R^2\chi - R\varepsilon \tag{4}$$

where $\theta = s/R$, and the prime denotes differentiation with respect to θ. The integral of eqn. (4) is

$$v(\theta) = A\cos(\theta) + B\sin(\theta) + \\ + \int_{\theta_0}^{\theta} \sin(\theta - \omega)\big(-R^2\chi(\omega) - R\varepsilon(\omega)\big)d\omega \tag{5}$$

where A and B are constants. In turn,

$$u = -v' + R\varphi \quad \text{and} \quad \varphi = C - R\int \chi(\omega)d\omega \tag{6}$$

where C is a constant (Belluzzi, 1934).

In the entire elastic domain (with the exception of the limit points A and F) each of the relations (3) can be inverted. So in regions B, C, D and E we can obtain explicit expressions for the kinematics parameters ε_0 and χ as functions of the internal axial force N and bending moment M. When the arch is statically determinate, the distribution for $N(\theta)$ and $M(\theta)$ are known, and the end conditions in terms of displacement allow for determining constants A, B and C. In the more general case of a statically indeterminate arch, instead, it is not known *a priori* in what zones the behaviour is linear and where it is non-linear, and the set of non-linear governing equations (3), (5) and (6) must be solved numerically through an iterative procedure (Bennati and Barsotti, 2001).

However, for simple geometric and loading conditions, such as flat arches or depressed arches with clamped sliding ends, the solution can be found explicitly: in fact, it is possible to write explicit expressions for the displacements and rotations. Moreover, the boundaries of the regions which behave non-linearly can be determined analytically.

The case of a masonry flat arch with clamped sliding ends

Let us consider a masonry flat arch AB, with clamped sliding ends, loaded by a transverse load q, uniformly distributed over its length l, and by an end compressive force P, as shown in Figure 3.

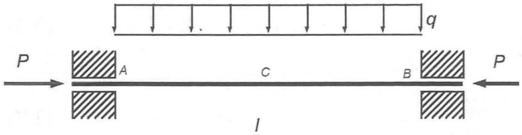

Figure 3
The flat arch

Let us keep the axial load P fixed, and let the distributed load q increase from zero. For low values of q, the behaviour of the flat arch is linear. At any given section, the point corresponding to the axial force (constant and equal to – P) and the bending moment in the (N, M) plane belong to region **E** of the elastic domain (Fig. 2).

If the abscissa x defines the distance of any given cross section from the end A, for increasing load q, the distribution of stresses will at first be non-linear in the two portions of the beam near the clamped ends A and B, where $0 \leq x \leq x_1$ and $x_2 \leq x \leq l$ (Fig. 4). In these two parts, the axial force and the (negative) bending moment fall into region **D** or **B** of the elastic domain, depending upon the value of the axial force. For small axial loads, the non-linearity concerns the tensile stresses (region **D**), while for high axial loads it will concern the compressive ones (region **B**).

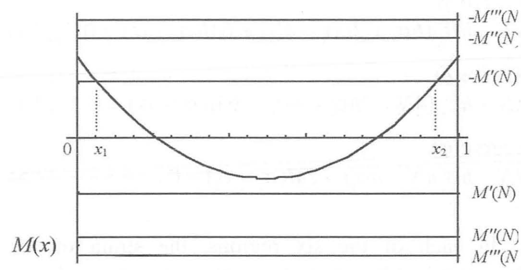

Figure 4
Bending moment diagram (the non-linearity concerns the two portions of the beam near the clamped ends)

In the lateral part of the flat arch, where the stress distribution is non-linear, the expressions for the rotation can be written as:

$$\varphi_D^-(x) = \varphi_d^-(x) + \overline{\varphi}_D^- \quad (0 \leq x \leq x_1 \text{ and } x_2 \leq x \leq l) \tag{7}$$

or

$$\varphi_B^-(x) = \varphi_b^-(x) + \overline{\varphi}_B^- \quad (0 \leq x \leq x_1 \text{ and } x_2 \leq x \leq l) \tag{8}$$

where $\overline{\varphi}_D^-$ and $\overline{\varphi}_B^-$ are constants determined by imposing the constraint conditions, while

$$\varphi_d^-(x) = \frac{8}{9} \cdot \frac{(P+\sigma_t h)^3}{E} \cdot \left\{ \begin{array}{l} -\dfrac{l-2x}{(2M_A + Ph + \sigma_t h^2 + qlx - qx^2)(8M_A + 4Ph + 4\sigma_t h^2 + ql^2)} + \\ -\dfrac{4}{\sqrt{q} \cdot (-8M_A - 4Ph - 4\sigma_t h^2 - ql^2)^{3/2}} \cdot \\ \cdot \arctan \dfrac{\sqrt{q}(l-2x)}{\sqrt{-8M_A - 4Ph - 4\sigma_t h^2 - ql^2}} \end{array} \right\} \tag{9}$$

and $\varphi_b^-(x)$ has an analogous expression, omitted here for brevity. Of course, the choice of expression (7) or (8) depends on the value of the normal force N (i.e., whether the point (N,M) of the elastic domain belongs respectively to region **D** or region **B**).

Since the bending moment $M(x)$ is a known function of x, the abscissas x_1 and x_2 (which correspond to the limit negative value of the bending moment, denoted in figure 4 by $-M'$, whose intensity is given by (2.1) or (2.2)) can easily be expressed in terms of both the applied loads and the redundant end restraining couple M_A.

As the structure is symmetric, the boundary and continuity conditions are:

$$\left\{ \begin{array}{l} \varphi_D^-(0) = 0 \\ \varphi_D^-(x_1) = \varphi_{lin}(x_1) \\ \varphi_{lin}\left(\dfrac{l}{2}\right) = 0 \end{array} \right. \quad \text{or} \quad \left\{ \begin{array}{l} \varphi_B^-(0) = 0 \\ \varphi_B^-(x_1) = \varphi_{lin}(x_1) \\ \varphi_{lin}\left(\dfrac{l}{2}\right) = 0 \end{array} \right. \tag{10}$$

depending on the value of the axial force N, where $\varphi_{lin}(x)$ is the expression for the rotation for $x_1 \leq x \leq x_2$, calculated with the bending moment and axial force belonging to the region **E** of the elastic domain. The three equations (10) form a non-linear algebraic system, whose solution allows determining the constants $\overline{\varphi}_{lin}$ and $\overline{\varphi}_D^-$ (or $\overline{\varphi}_B^-$) and the redundant end restraining couple M_A.

The non-linear equilibrium solution determined by system (10) holds until the bending moment in the middle of the beam reaches the border of region **E** of the elastic domain.

If we increase the load q, the non-linear distribution of stresses will also involve the central portion of the beam, that is for $x_2 \leq x \leq x_3$ (Fig. 5), in which the normal force N and the positive bending moment M will once again belong to the regions **D** or **B**, depending on the value of the constant axial force N.

The respective explicit expressions for the rotation are then:

$$\varphi_D^+(x) = \varphi_d^+(x) + \overline{\varphi}_D^- \qquad (x_2 \leq x \leq x_3) \tag{11}$$

or

$$\varphi_B^+(x) = \varphi_b^+(x) + \overline{\varphi}_B^- \qquad (x_2 \leq x \leq x_3) \tag{12}$$

where $\overline{\varphi}_D^+$ and $\overline{\varphi}_B^+$ are unknown constants, while

$$\varphi_d^+(x) = \frac{8}{9} \cdot \frac{(P+\sigma_t h)^3}{E} \cdot \left\{ \begin{array}{l} -\dfrac{l-2x}{(2M_A - Ph + \sigma_t h^2 + qlx - qx^2)(8M_A - 4Ph - 4\sigma_t h^2 + ql^2)} + \\ -\dfrac{4}{\sqrt{q} \cdot (-8M_A + 4Ph + 4\sigma_t h^2 - ql^2)^{3/2}} \cdot \\ \cdot \arctan \dfrac{\sqrt{q}(l-2x)}{\sqrt{-8M_A + 4Ph + 4\sigma_t h^2 - ql^2}} \end{array} \right\} \tag{13}$$

and $\varphi_b^+(x)$ has an analogous expression, omitted here for brevity.

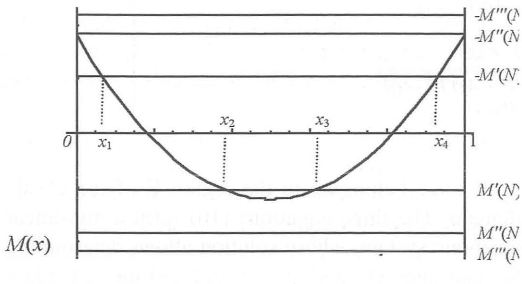

Figure 5.
Bending moment diagram (the non-linearity concerns the central portion and the two portions of the beam near the clamped ends)

If we increase the load q, while maintaining P constant, the non-linearity of the stress distribution will concern both the tensile stresses and the compressive ones first in the two lateral portions and subsequently also in the central part of the beam. In these beam portions , that is for $0 \leq x \leq x_1$, $x_8 \leq x \leq l$ and $x_4 \leq x \leq x_5$ (Fig. 6), M will belong to region **C** of the elastic domain; it will therefore be bounded by values $M''(N) \leq |M| \leq M'''(N)$ (where $M''(N)$ is given by (2.3) or (2.4), depending on the value of the axial force N, whereas $M'''(N)$ is given by (2.5)). The rotations in these two regions are respectively:

$$\varphi_C^-(x) = \varphi_c^-(x) + \overline{\varphi}_C^- \qquad (0 \leq x \leq x_1 \text{ and } x_8 \leq x \leq l) \tag{14}$$

$$\varphi_C^+(x) = \varphi_c^+(x) + \overline{\varphi}_C^+ \qquad (x_4 \leq x \leq x_5) \tag{15}$$

and

where $\overline{\varphi}_C^-$ and $\overline{\varphi}_C^+$ are once again constants determined by imposing the constraint conditions, while

$$\varphi_c^-(x) = -\frac{\left(-(\sigma_c - \sigma_t)\right)^{3/2}}{2E\sqrt{3q}} \cdot \arctan\left[\frac{\sqrt{-3q(\sigma_c - \sigma_t)(l - 2x)}}{2\sqrt{-6\left(M_A + \frac{ql}{2}x - \frac{q}{2}x^2\right)(\sigma_c - \sigma_t) - 3(P + \sigma_t h)(P + \sigma_c h)}}\right] \tag{16}$$

and

$$\varphi_c^+(x) = -\frac{\left(-(\sigma_c - \sigma_t)\right)^{3/2}}{2E\sqrt{3q}} \operatorname{Log}\left[\begin{array}{l} -\sqrt{-3q(\sigma_c - \sigma_t)(l - 2x)} + \\ +2\sqrt{\begin{array}{l} 6\left(M_A + \frac{ql}{2}x - \frac{q}{2}x^2\right)(\sigma_c - \sigma_t) + \\ -3(P + \sigma_t h)(P + \sigma_c h) \end{array}} \end{array}\right] \tag{17}$$

In all the cases previously described, the solution can be found by imposing the restraining conditions (for example, that the rotation be nil at the clamped ends when the intensity of the bending moment at A and B is less than $M'''(N)$) and the continuity condition at the contact points between the different zones along the flat arch where the constitutive behaviour is different.

Finally, it is also possible to determine the vertical displacements by imposing that they be nil at the clamped ends. For the sake of brevity, their explicit expressions have been omitted here.

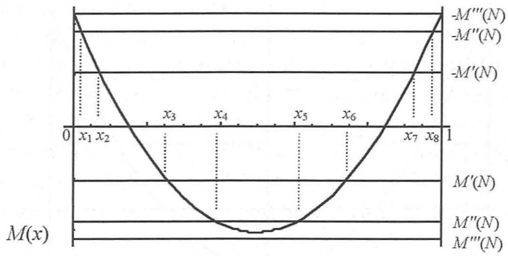

Figure 6
Flat arch: bending moment diagram (the non-linearity of the stress distribution concerns both the tensile and compressive stresses in the two lateral portions and the central part of the flat arch)

The case of a depressed circular arch with clamped sliding ends

Let us now consider a depressed circular arch ACB with clamped sliding ends A and B, whose span, radius and centre angle are, respectively, l, R and 2α, with *small l/R*. We assume that this arch is subjected to a vertical load q, uniformly distributed over its span l, and a compressive concentrated load P at its ends, as shown in Figure 7.

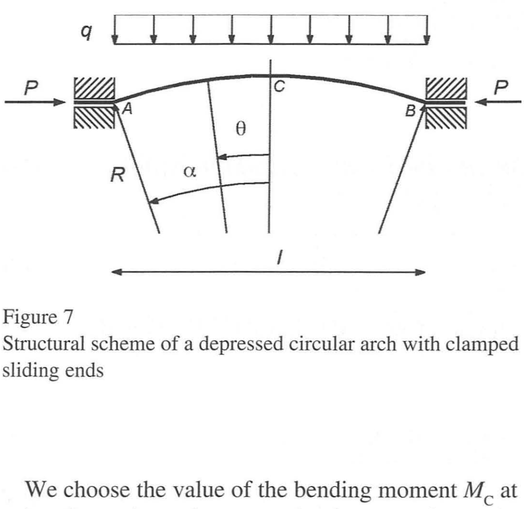

Figure 7
Structural scheme of a depressed circular arch with clamped sliding ends

We choose the value of the bending moment M_C at point C as the unknown redundant reaction. The formal expression for the bending moment can be written as a function of the unknown moment M_C:

$$M(\theta) = M_C + PR(1 - \cos\theta) - \frac{qR^2}{2}\sin^2\theta. \qquad (18)$$

In order to find this unknown reaction, it is necessary to impose the constraint conditions. We impose that the rotation φ (θ) be nil? at the two sections A and C, respectively identified by $\theta = \alpha$ and $\theta = 0$.

Since the values of the angle θ are *sufficiently small*, the axial force can be considered constant and the expression for the bending moment can be simplified by developing (18) according to a Taylor series up to the second order:

$$N(\theta) = -P \qquad M(\theta) = M_C + (PR - qR^2)\frac{\theta^2}{2}. \qquad (19)$$

When the axial force and bending moment belong to the region **E** of the elastic domain, the solution is linear elastic and the bending moment at the keystone is

$$M_C = -PR\frac{\alpha - \sin\alpha}{\alpha} + \frac{qR^2}{4}\frac{\alpha - \sin\alpha\cos\alpha}{\alpha}. \qquad (20)$$

If we increase the load q, while leaving the axial force $-P$ constant, the distribution of stresses will be non-linear at first in the two portions of the arch near the clamped ends A and B, where the non-linear distribution will concern the tensile stresses or the compressive stresses, respectively depending on whether N and M belong to the region **D** or **B**. Then, the central portion of the arch will also start exhibiting non-linear behaviour (with N and M belonging to the regions **D** or **B**). If the load q still increases, while P is always constant, the non-linearity of the stress distribution will concern both the tensile and compressive stresses in the two lateral portions and subsequently extend to the central part of the arch as well. In these portions of the arch, the bending moment M and axial force N will belong to the region **C** of the elastic domain (Fig. 8).

With reference to Figure 8, the explicit expressions for the rotation φ are:

$$\varphi_C^+(\theta) = \varphi_c^+(\theta) + \bar{\varphi}_C^+$$

$$\text{for } \theta_4 \leq \theta \leq \theta_5 \qquad (21)$$

$$\varphi_D^+(\theta) = \varphi_d^+(\theta) + \overline{\varphi}_D^+(\theta) \text{ or } \varphi_B^+(\theta) = \varphi_b^+(\theta) + \overline{\varphi}_B^+(\theta)$$
$$\text{for } \theta_3 \le \theta \le \theta_4 \text{ and, } \theta_5 \le \theta \le \theta_6 \tag{22}$$

$$\varphi_D^-(\theta) = \varphi_d^-(\theta) + \overline{\varphi}_D^-(\theta) \text{ or } \varphi_B^-(\theta) = \varphi_b^-(\theta) + \overline{\varphi}_B^-(\theta)$$
$$\text{for } \theta_1 \le \theta \le \theta_2 \text{ and, } \theta_7 \le \theta \le \theta_8 \tag{22}$$

depending on the value of the axial force, and

$$\varphi_C^-(\theta) = \varphi_c^-(\theta) + \overline{\varphi}_C^-$$
$$\text{for } -\alpha \le \theta \le \theta_1 \text{ and } \theta_8 \le \theta \le \alpha \tag{24}$$

In (21–24), $\overline{\varphi}_D^-$, $\overline{\varphi}_B^-$, $\overline{\varphi}_D^+$, $\overline{\varphi}_B^+$, $\overline{\varphi}_C^-$, and $\overline{\varphi}_C^+$ are constants to be determined by imposing the restraining conditions, while, for example,

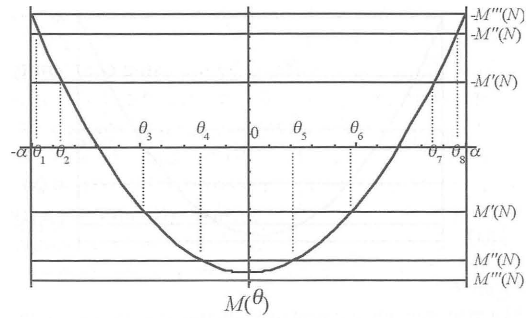

Figure 8
Depressed arch: bending moment diagram near the collapse condition

$$\varphi_d^+(\theta) = \frac{8R(-P - \sigma_t h)^3}{9E} \cdot \left(\begin{array}{l} \dfrac{\theta}{2(2M_C - Ph - \sigma_t h^2)(2M_C - Ph + PR\theta^2 - qR^2\theta^2 - \sigma_t h^2)} + \\ + \dfrac{\arctan\left(\dfrac{\theta\sqrt{qR^2 - PR}}{\sqrt{-2M_C + Ph + \sigma_t h^2}}\right)}{2\sqrt{qR2 - PR}(-2M_C + Ph + \sigma_t h^2)^{3/2}} \end{array} \right) \tag{25}$$

$$\varphi_d^-(\theta) = \frac{8R(-P - \sigma_t h)^3}{9E} \cdot \left(\begin{array}{l} \dfrac{\theta}{2(2M_C + Ph - \sigma_t h^2)(2M_C + Ph + PR\theta^2 - qR^2\theta^2 + \sigma_t h^2)} + \\ + \dfrac{\operatorname{arctanh}\left(\dfrac{\theta\sqrt{qR^2 - PR}}{\sqrt{2M_C + Ph + \sigma_t h^2}}\right)}{2\sqrt{qR2 - PR}(-2M_C + Ph + \sigma_t h^2)^{3/2}} \end{array} \right) \tag{26}$$

$$\varphi_c^+(\theta) = -\frac{1}{2E}\sqrt{-\frac{R(\sigma_c - \sigma_t)^3}{2(qR - P)}}\,\mathrm{Log}\left(2\theta\sqrt{-3R(qR - P)(\sigma_c - \sigma_t)} + 2\sqrt{A(M(\theta), N(\theta))}\right) \tag{27}$$

and

$$\varphi_c^-(\theta) = -\frac{1}{2E}\sqrt{-\frac{R(\sigma_c - \sigma_t)^3}{3(qR - P)}}\,\arctan\left(-2\theta\sqrt{-3R(qR - P)(\sigma_c - \sigma_t)} \,/\, \left(2\sqrt{B(M(\theta), N(\theta))}\right)\right) \tag{28}$$

with

$$A\big(M(\theta), N(\theta)\big) = 6M(\theta)(\sigma_c - \sigma_t) - 3\big(N(\theta) - \sigma_t h\big)\big(N(\theta) - \sigma_c h\big)$$

and

$$B\big(M(\theta), N(\theta)\big) = -6M(\theta)(\sigma_c - \sigma_t) - 3\big(N(\theta) - \sigma_t h\big)\big(N(\theta) - \sigma_c h\big)$$

The other rotations have analogous expressions, omitted here for brevity.

The solution can be found by imposing conformity to the restraining conditions and the continuity condition between different zones, each characterised by a different constitutive relation. Once the value of the redundant reaction M_C is known, it is also possible to determine the displacements $u(\theta)$ and $v(\theta)$ by imposing the proper end conditions. For the sake of brevity, their explicit expressions have been omitted here.

It is worth noting that, in order to make it clear how the tensile strength affects the solution, all the expressions for the rotations listed above have been obtained in the general case of σ_t different from zero.

For given values of the axial force P and uniformly distributed load q, the equilibrium problem leads to a non-linear algebraic system of equations in the integration constants which can be solved in a semi-explicit way. By way of example, figures 9, 10, 11 and 12 show some results obtained by setting $\sigma_t = 0{,}3$ MPa; $\sigma_c = 20$ MPa; $E = 7{,}000$ Mpa; $h = 100$ cm; $l = 1{,}000$ cm; $\alpha = 0.1$. 7.000
1.000

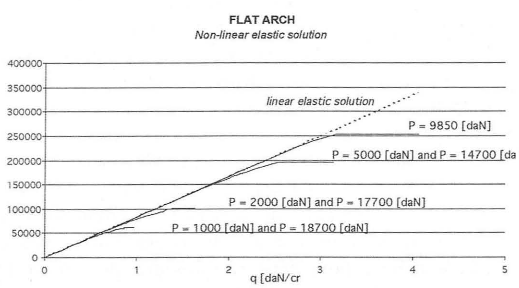

Figure 9
Flat arch: redundant reaction M_A vs uniformly distributed load q for different values of the axial force $N = -P$

With reference to figure 9, it is interesting to observe that the flat arch exhibits its maximum load bearing capacity for $P = -h(\sigma_c + \sigma_t)/2 = 9850$ daN.

As one can see, the rotations at the end sections A and B and the vertical displacement at the middle

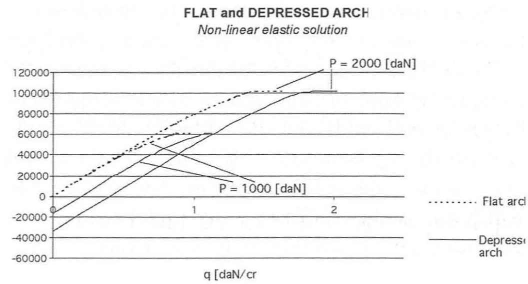

Figure 10
A flat and depressed arch with the same span and height: redundant reaction M_A vs uniformly distributed load q

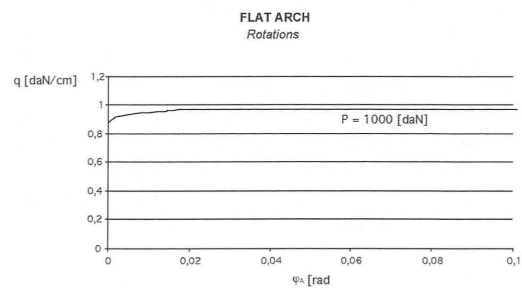

Figure 11
Flat arch: uniformly distributed load q vs rotations φ_A at the ends

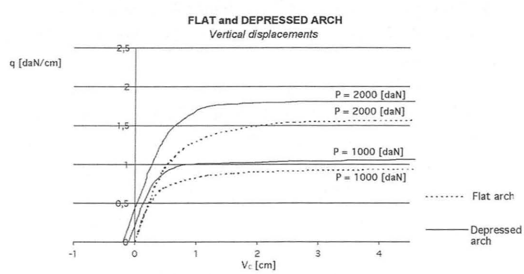

Figure 12
A flat and depressed arch with the same span and height: uniformly distributed load q vs vertical displacements v_C at the midpoint C

section C diverge as load q approaches from below a threshold value, which can be determined by the limit analysis. This corresponds to the development of a collapse mechanism characterised by the formation of hinges at both ends and the midpoint. Such results suggest the advisability of defining a conventional value of the collapse load as that in correspondence to which the residual stiffness of the arch becomes less than some pre-set fraction of the initial value.

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The first generation of reinforced concrete building type. A public housing estate in Lisbon

Alexandra Alegre
Teresa Heitor

The paper aims at analysing the building process carried on a housing estate, Alvalade, in Lisbon during the 1940's. Particular concern was given to the construction system and to the appliance of new building technologies. It comprises 302 buildings performing the first generation of reinforced concrete building type: dual structures with masonry resistant walls, wooden pavements and reinforced concrete slabs on wet areas (kitchen and W. C.), heavily infilled with brick walls. Pre fabricated components were also introduced as part of the innovative experience (e. g. doorsteps, window frames, wall cover). The operation began with an experiment contract work involving 3 buildings to test the building procedures and to ensure its success.

The estate is part of a planned intervention —decided, conducted and financed by the Central Government, aiming at offering affordable living space within a short time for public renting. It has a consistency in character, which stems from policies, building regulations and cost limitations formulated in the master plan. It is characterised by a disciplined urban and architectural design approach. The urban layout is arranged according to clusters, based on the primary school in the heart. Built-up fabric is geometric in character and building typologies, materials and forms are strictly limited. The plain architecture —self-contained 4-storey flats, placed in blocks of regular shape— with its studied detailed is a cultural declaration to simplicity. Dwelling spatial layouts define functional sectors according to a rational use of domestic space and family needs.

Nowadays the area is under a gentrification process. Buildings show performance problems due to functional and constructive anomalies as well as to unsuited renewal works made by the householders without appropriate knowledge of the construction system.

The paper considers three parts. The first one introduces the case study. It is focused on the social, political and urban context that worked as background to the conception of this area. The second part describes in detail the built up fabric and the building process. It refers to the spatial, physical and functional features and to construction system. The third part considers the present situation. Functional obsolescence indicators and constructive anomalies are identified and renewal interventions are analysed.

Introduction

Alvalade corresponds to the first significant urban operation planned to expand Lisbon urban fabric by public initiative during the second quarter of the XX century.

This new area was destined to the construction of infra structures, housing and public facilities. It was programmed to integrate social housing as well as low rental housing, supported in equipments —school, market, civic centre— and in small industry.

The integration of small scale building typologies (4 storey; 2 flats per floor) and innovative ways of housing promotion made the venture financially viable, diversifying housing supply in such a way as to include market values and allowed for the construction of a diversified and multi functional social fabric. The use of then innovative processes both at the level of conception —a project type with twelve variants— as well as at the level of construction and management —experimental construction— permitted the success of this operation.

Figure 2
Alvalade Master Plan: spatial layout

The master plan

Alvalade Master Plan was planned at the start of the 40's. Framed within the structural scheme of Lisbon, proposed in the *City master plan*, elaborated by Etienne de Gröer (1938–1948), promotes the northward expansion of the city, and largely gives response to the elimination of the housing deficit that affected the city.

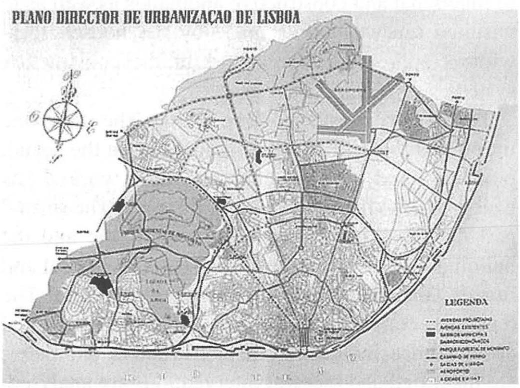

Figure 1
City master plan (1938–1948)

Alvalade occupies an area of about 230 acres. It is formed by a rectangular hierarchical grid, divided by a net of main axes defining eight cells, thus creating «neighbourhood units». The implementation of the plan created 12.000 houses for a population of 45.000. The first set of 84 buildings was inaugurated in 1947 (CML 1948)

In its organization the plan mirrors concepts and influences that characterized the first decades of the twentieth century. It applies concepts of modern urbanism such as neighbourhood units, the distributive allocation of functions and equipments, the hierarchy of the of the road system through avenue, streets, pedestrian paths, the de-privatisation of the ground and the freeing the inner part of the blocks for collective use.

The cells, «housing units», are structured from a central element, the primary school, around which the housing is distributed. Its average dimension was calculated in such a way as not to exceed a distance of 500 meters from school to house. The connection between these two points is facilitated by the creation of pedestrian paths that cross the inner space of the housing blocks.

These are designed as large common exteriors, to be used by the dwellers (CML 1948). The equipments and the large recreational spaces are distributed in such a way as to be accessible by the dwellers of each cell through comfortable and short paths, which occasionally cross the main arteries, where there is rapid transit. Within each cell the streets are the local access to buildings.

Low rental housing

Characteristics of Low Rental Housing

Alvalade started with the construction of the programme of low rental housing located in cells I

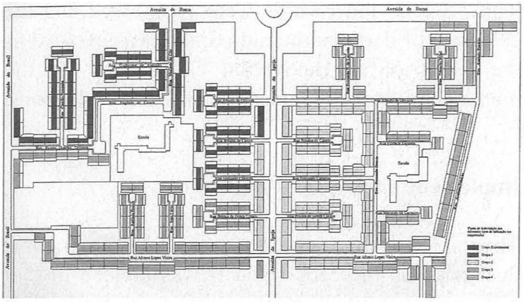

Figure 3
Low Rental Housing in cells I and II of Alvalade

and II of the master plan. It considered the construction of 2.066 flats in 302 buildings of three and four floors without elevator.

The study of low rental housing under the guidance of Architect Jacobetty Rosa, considered the development of 9 types of housing grouped in three series of three types each, corresponding to the families' different social levels and the number of persons per house.

At a level of functional organization, in each series the types differ according to the introduction of a bedroom, and the variation series to series is achieved through the introduction of a den office and larger areas for complementary installations from series I to series II; and with the introduction of

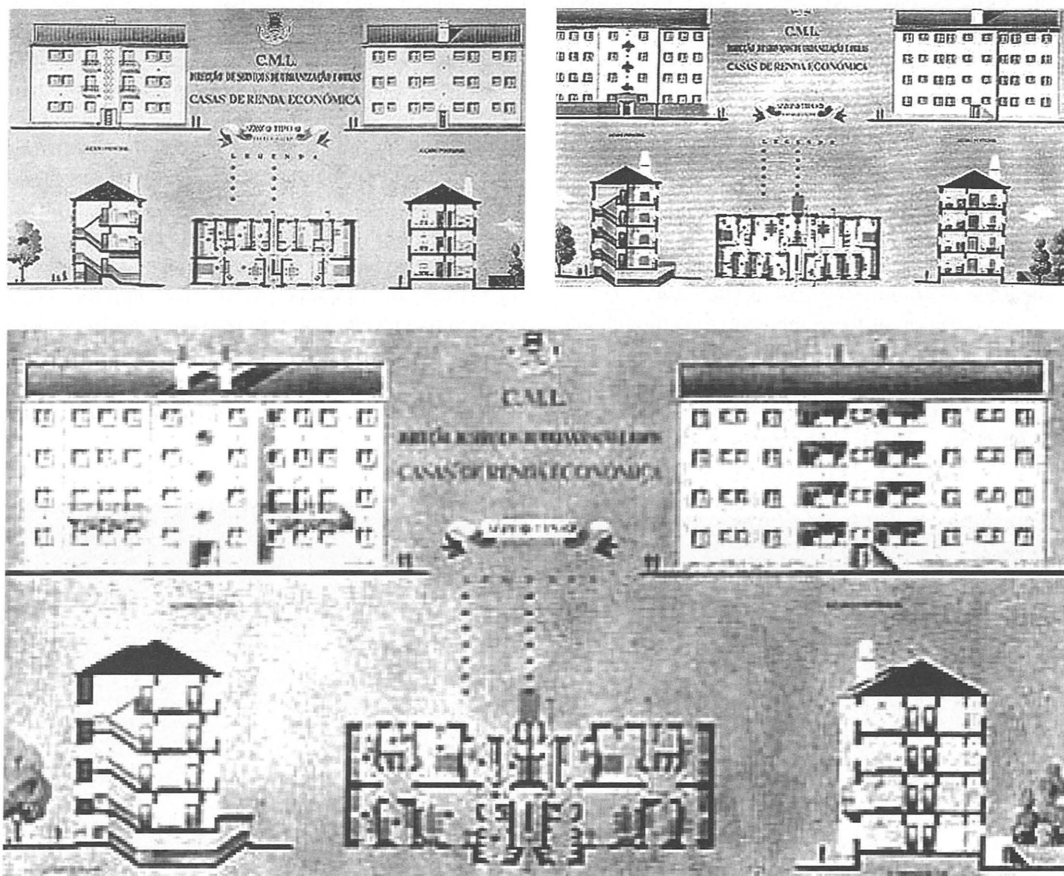

Figure 4
Type Records of series I (Type 3), series II (Type 6), and series III (Type 9) with plans, cross section and elevation

installations for a maid (room with sanitation) from series II to III.

The buildings fit in a bloc typology, with a rectangular plan. They were grouped in open blocks, allowing the existence of small gardens in the backyards. Formally, the buildings are characterized by sobriety and the small variation of the elements of architectonic composition; thus the housing estate presents an exterior image of great unity.

The plans of the different housings is an economic solution based on the rationalization of the spatial organization and from the adaptation of spaces to the designated functions: an increase of the liveable area of each house through the reduction of unused spaces (elimination of the corridor integrating its area in the living room) and the creation of simple and rational plans; further useful areas were obtained by inserting furniture in the construction.

The rationalization of the housing was based on the study and analysis of meaningful indexes that permitted to perfect the correlation conditions of the different housing functions, and so adopt distribution solutions of the different partitions that allow for greater comfort in domestic life. Above all, it is aimed at rationalizing the house in such a way as to «increase the value of the house reducing to the compatible minimum its area» (CML 1946).

The application of these innovative methodologies to the concern over creating housing with good habitability conditions —natural light in all the partitions, transversal ventilation of the apartments and hygiene— yields apartments with rectangular plans. So the inner yards were suppressed and the existence of dark and humid corners was avoided as the result of an innovation carried out by the municipality in the organization of the housing plans.

Implementation of the Programme

The implementation of the 302 buildings programme that make up the cells I and II was divided in four construction contract works, with a similar load of work, added to the construction of an experimental group made by 3 buildings of different types (Types 3, 6 and 8). This option allowed to test the solutions adopted in the projects, both from the point of view of the conception as well as the constructive technologies; thus providing useful lessons for the overall implementation of the programme. The cost of construction of this experimental group, while being superior to the expenses forecasted, became self-rewarding in as much as it allowed to introduce corrections in the construction of the following houses. In the first three construction phases stone and brick masonry was used, the fourth is different for the use concrete block masonry. The rationalization and simplification processes applied to the study of housing were also applied at a constructive level, thus applying solutions that seek to concentrate kitchen and sanitary plumbing, a rigorous choice of materials and construction processes, as well as a careful planning and management of the construction.

Figure 5
Low rental housing. Conclusion of the first group

Figure 6
Aerial view of cells I and II of the urbanization Plan of Alvalade: execution of the different construction groups

Together with the construction phases, supplies of materials and construction elements were organized through out time periods according to the general plan of works, in such a way as to guarantee regularity in supply. Thus there were independent programmes for the supply of wood, fenestration and doors, aiming at an improvement of the quality of these elements; an action programme for the supply of plumbing, concrete tiles, sanitation equipment, bricks and roof-tiles.

The standardization and pre-fabrication of some elements used in the construction allowed for reduced costs and swifter construction: pre-fabricated elements such as doors windows and cladding; standard concrete elements for steps and exterior windows lintels and sills of standard dimension, standard tubes for water supply, drainage and electricity.

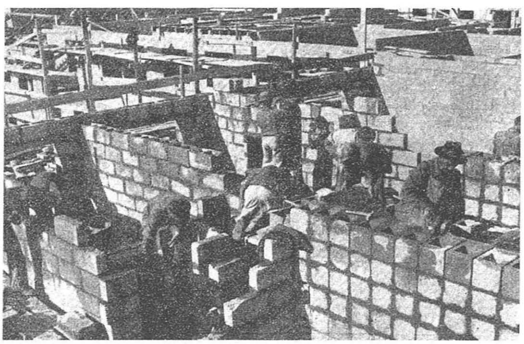

Figure 7
Execution of walls of concrete block masonry

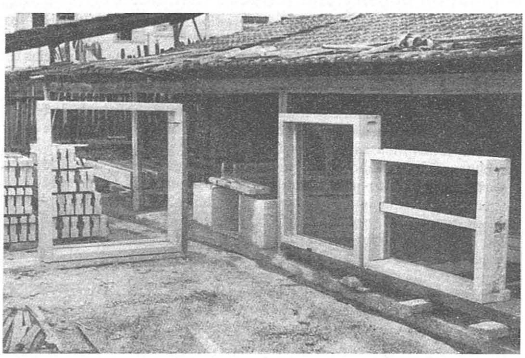

Figure 8
Standard concrete elements: exterior windows lintels and sills

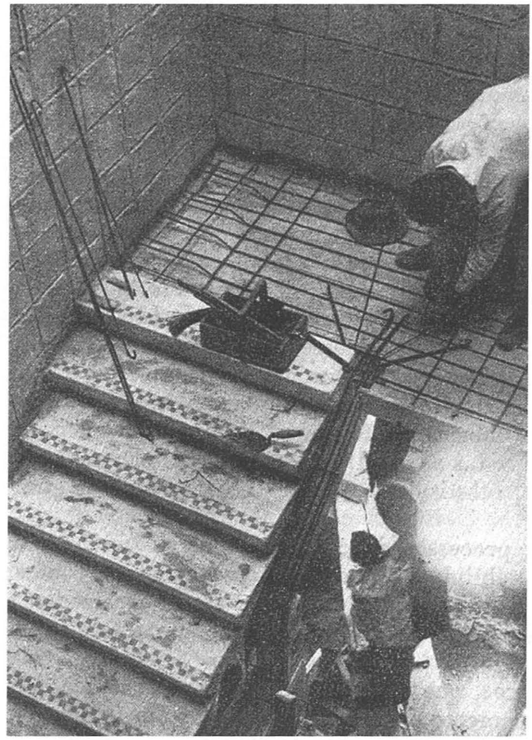

Figure 9
Standard concrete elements: steps of staircase

At the planning and management level of the construction, municipality counted with the help of the «Federação of the Caixas da Previdência», that by the end of 1946, in a contract signed with the Lisbon municipality, made available the financing needed to start the Low rental construction programme. According to this contract it was up to the Caixas de Previdência to: pay expenses of the contractors and builders; expenses with suppliers and unexpected expenses, up to 5% of those expenses, and expenses relative to the supervision and control of the works, at a 1% of the total of remaining expenditure. On its turn, it was up to Lisbon City Hall to: elaborate the overall urbanization plan, including all the technical economical and social studies; elaborate the general plan of the works, including the definition of deadlines for execution and the programmes for the

different stages of construction; to organize the special actions to assure the supply of all the necessary construction materials; to assign according to a public auction, the production of pre-fabricated elements, as well as supervise and guide its production.

Throughout the construction process, various tests of materials and execution processes that were at the time innovative in terms of construction management. Specifically one notes the testes of concrete without fine sand o; the studies on the most adequate mortar of concrete for application in interior and exterior cladding and for the elaboration of masonry, particularly the concrete blocks; the studies elaborated by the National Laboratory of Civil Engineering, relating to plaster and woods. For the latter it was proposed a more adequate treatment for this material, one that permitted better quality in fenestration.

It was also aimed through the different construction phases, to improve the quality of the ironworks used creating more functional and economical types; the grés tubes, the sanitation equipment and the hydraulic tiles satisfied predefined requirements for quality and fabrication. Employing concrete-block masonry rather than stone and brick lead the municipality to acquire an installation for the manufacture of the blocks, thus trying to satisfy the technical and economic conditions to stabilize production.

Constructive characterization

The buildings of the cells I and II belong to an era of construction that characterized the first half of the twentieth century in Portugal —the transition to reinforced concrete. As mentioned above, its execution was divided in four groups of contracted works.

The first three are characterized by the use of stone masonry (hydraulic masonry) in the exterior walls and brick in the inner walls, belted on all floors by reinforced concrete beams at the height of the window lintels. The pavements in the kitchens and W. C.s were built with slabs of reinforced concrete, the rest were made with wood beams and Portuguese floorboard. The ladders were built with prefabricated concrete steps, supported by brick fronts.

The fourth is characterized by the use of hollow concrete blocks in the outer walls, by massive concrete blocks in the front walls and by stone or plaster in the partition walls. The pavements were executed with pitch blocks that made coffering unnecessary.

In all the building projects the roofs are made with a wood structure and tiling in Lusa roof tiles. The fenestrations are pinewood and the outer walls plastered with concrete mortar of and sand, and finished in the exterior with mass of sand, and in the interior plastered.

It is worth referring that the execution of the replacement of the wood pavements for reinforced concrete, in buildings that presented serious anomalies in the wood used in the execution of those elements. These works were carried during the fifties.

Conclusion

The exit of Alvalade experiment is reflected in the great advantages of an urbanization orderly planned, studied and executed by the municipality; that established rule both at the urban planning and design as well as the conception and construction of housing. This experience also permitted to demonstrate that construction costs can be controlled through the study of the technical conditions of execution: at the level of materials, use of standard and pre-fabricated elements and construction solutions that are economically viable.

However, after the conclusion of the urbanization some deficiencies were noted: a slight excess of population density and reduced distance between the facades in the secondary lanes, excessive distance between some housing and the commercial area of the neighbourhood, inconvenience of the inner patio spaces, individualized and walled, seldom used by residents.

Almost fifty years after the conclusion of the urbanization, this now appears as a consolidated area of Lisbon, with urban, constructive and architectonic characteristics that make it distinct from the rest of the city. Through out this period there are several post occupancy changes at various levels: At a physical level, in the buildings that constitute the neighbourhood; at a social level in the inhabitants; at a functional level, both in the built constructions as in the urban space.

At a physical level the buildings sustain a level of degradation resulting from the wearing and aging of materials, associated to the effects of pollution and weather combined with an absence of maintenance. There are some occasional actions of executed inside the buildings that may aggravate their maintenance condition. At a social level, Alvalade has witnessed a process of 'gentrification' involving new residents with new dwelling habits. The social position of the population is characterized by the new composition of families, with smaller numbers and greater purchasing power. This «new» residents are responsible for the alterations introduced to housing, adapting it to new functional requirements, with an impact in the spatial and constructive organization, resulting in interventions that, because of their depth and lack of knowledge of the built reality are responsible for negative consequences in the construction and structure of the buildings.

At the urban space level, one notes some environmental problems, namely: security, parking, public lightning, illegal and disorderly of the inner accessible spaces —spaces that were planned to be green and recreational for the benefit of residents.

Alvalade is the first experience of integral, defining norms and rules at an urban, architectonic and constructive level. The elaboration of a municipal regulation that established specific rules of action, guaranteeing the goals of an urban rehabilitation programme is imperative. This process must be supported in its totality by an knowledgeable, multidisciplinary team in the area of rehabilitation, Thus, it is guaranteed the aesthetical and urban unity of the built ensemble, that not only occupies a noble area of the city of Lisbon, but is also representative of a well defined urban development phase, with its own and exceptional characteristics.

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The grandstand roof of the Zarzuela Hippodrome in Madrid: The constructive talent of Eduardo Torroja

Joaquín Antuña Bernardo

Introduction: The construction of the New Race track of Madrid

The old royal race track of Madrid, located in the north end of the Paseo de la Castellana, was demolished in 1933. To build a new one, the Gabinete Técnico de Accesos y Extrarradio de Madrid summoned, on July 6 1934, a design competition. The new construction would be located in the mount of «El Pardo», property of Patrimonio de la República, where they were being carried out the tracks of careers. The proposals should be adjusted to this layout.[1] Nine projects of architects or engineer/architect teams were presented, and the verdict of the jury became public December 18 1934.

The magazine *Hormigón y Acero*, edited by the engineers E. García Reyes and E. Torroja, dedicated the number of November 1934 to the design competition, publishing an article of each one of the authors that presented a proposal to the competition, in which they explain their project.

The project for the new race track included several buildings also the stands, like stables and employees' housings, as well as the urbanization of the environment, organizing the parking areas and the circulations. Overall it was an extremely complex program, as illustrated by the width and depth of the jury's verdict. The building of the stands was the most representative construction in each proposal, furthermore being the one that had a more complex program of uses, and in the one that the solution of the roof of the tier was the most outstanding structural problem.

The Arniches-Domínguez-Torroja proposal

The Carlos Arniches, Martin Domínguez and Eduardo Torroja team won with a proposal that does not coincide exactly with the one that was finally built, although the same distribution of buildings and its general aspect remains. The area of spectator stands consisted of three independent aligned buildings: the partners' stand of 30.00 m length, located among those of preferred and general admission (each 60.00 m long). These three blocks and the adjacent restaurant building were united at the level of the track for a continuous gallery, connected through an open arcade (figure 1).

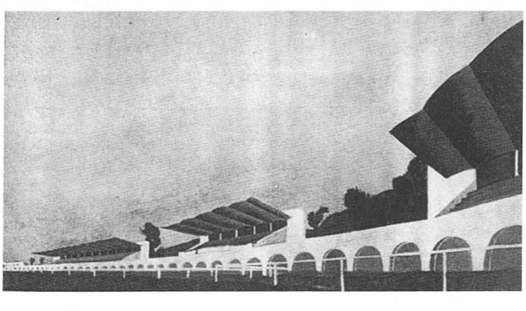

Figure 1
View of the proposal presented to the competition

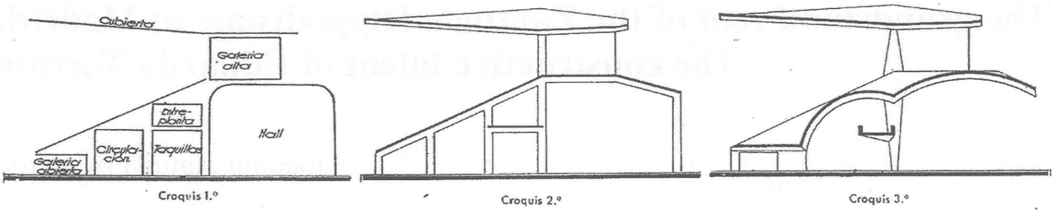

Figure 2
Sketch of the proposal

The grandstand structure is similar in all the proposals: a concrete frame with the roof formed by a cantilever of variable thickness supporting a solid slab. The organization of the transverse section of the buildings arises from the functional program, as the author shows in the outline 1° of Figure 2 below. The design progressed and that it can be materialized according to the outlines 2 and 3 of the figure 2. Comparing them with the proposal, the permanency of the roof type is appreciated, formed by some vaults supported on cantilevers. There is a contradiction between the 3° proposal (as built), in which there is no pier for the hall, and in Figure 3, showing an external pier.

The building consists, therefore, in a sloped stand (A in the figure 3), on which is formed the tier. Underneath there are two spaces located at different levels, one of them at the level of the tracks B, that is a roofed gallery, and other (C) at a higher level than the tracks and connected with the gallery by a stairway, where the box offices are located. The first space is the *«galería de pista»* (gallery), and the second the *«zona de taquillas»* (ticket zone). The roof of the tier continues forming a terrace (D), which roofs the hall (E), contiguous to the *«zona de taquillas»*. This space is called the *«sala de apuestas»*. In the space of *«zona de taquillas»* there is a gallery in passing (J) placed 2.00 m of the floor, is the *«galería de servicio»*. The tiers and the later terrace are protected by the roof (F) that supports in two elements, the support (G) and the truss (H). This arrangement allows the spectators as much space in

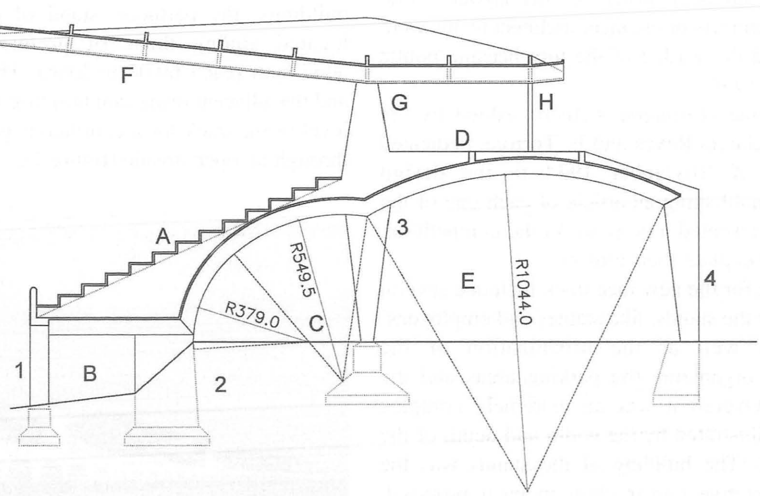

Figure 3
Transverse section of the initial proposal

the tier as in the track, to access the box offices easily. At the same time, a terrace is formed behind the stands from which one can see to both sides.

The structure of the grandstand in the proposal of the competition

The grandstand structure consists of a series of two-story concrete frames, the first floor is supported by four supports, and the roof on two supports. The first floor is formed by a beam of three spans, one horizontal among the supports 4 and 3, and another curved beam between the supports 3 and 2, with a horizontal span until the support 1. In this form the arcade closes the gallery at the track level.

The particularity of these frames, compared to the other proposals, is that the inferior face of the beams of the first plant is not straight, but rather has the form of two circular arch segments, of a larger radius in the area of the hall, and of a smaller radius in the betting area. Among these frames a slab of 6 cm of thickness was placed, reinforced by some ribs of 20 by 10 cm whose inferior face, in the longitudinal sense, has the form of a circumferencial arch. This gives place to a series of span of torus vaults among the frames.

The structure of the roof

The roof consists of a beam of variable edge supported at point G with a cantilever of 12,75 m span, and anchored in a truss (H) located 5.25 m behind the previous one, and another back cantilever of 1.00 m. Among these beams are a series of spans of cylindrical vaults of circular form. The vaults have a 5.00 m span, 55 cm rise and 6 cm of thickness. As stiffener, they are placed, each 2,45 m, some ribs of 20 × 10 cm section, which are not connected to the support beams (figure 4).

The horizontal thrust of each vault, for a uniform loading, is balanced with those of the contiguous span, giving only a vertical reaction on the cantilever. In the spans at the end of each roof it is necessary to prepare an element with enough rigidity in the horizontal plane to carry the thrust. This situation is solved by making the last half vault become a beam that, working in the horizontal plane, can resist the thrust of the previous section (figure 5). This

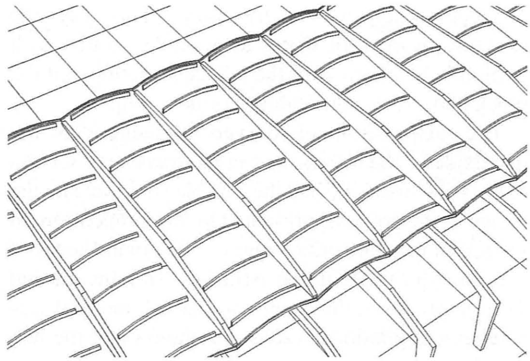

Figure 4
Aerial view of the proposal

horizontal beam is supported in the same supports as the beam of the last frame that, in this case, has to resist the corresponding vertical and horizontal reactions. However, that support has the same section as the others, and it lacks rigidity in the transverse sense. Therefore, to balance the thrust of the roof vaults and the floor vaults (of the same size), some transverse elements are needed to provide enough rigidity. To achieve this, vertical cylinders are placed in the ends of the buildings, whose lateral walls balance the horizontal thrust. We call these structural

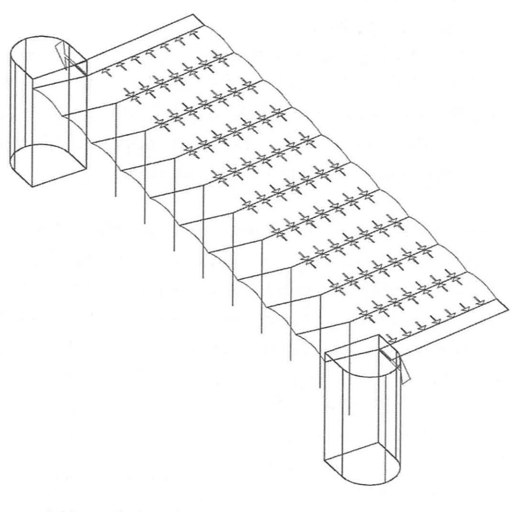

Figure 5
Outline of the horizontal forces in the roof structure

cylinders «*cuerpos de extremidad*». This way, the structure of the roof works as a whole, so that each section is balanced with the adjacent section and the block ends assure the stability of the group.

The disposition of curved elements, in the underside of the beams, and in the transverse section of the roof, distinguished the Torroja structure and the image of the proposal presented to the competition. In the rest of the competition entries, the grandstand roof was straight, while in the Arniches, Domínguez and Torroja proposal, this front was curved, the roof was a series of cylindrical vaults and the roof of the hall was formed by a succession of torus surfaces.

THE INITIAL PROJECT DIFFICULTIES

Between the verdict of the competition, published in December of 1934, and the definitive approval of the project in September of 1935, the authors modified the initial proposal varying, among other things, the roof construction solution. These changes transformed the execution process, eliminating some of the difficulties presented by the initial solution, but without altering the aspect of the buildings. The roof structure consisted of some cylindrical vaults, supported in parallel beams. The formal characteristics of the roof can be summarized as:

a) from the tier, the roof leaves like a series of parallel vaults;
b) the beams that support the roof are inclined, rising from the support toward the ends;
c) the vaults have the same depth in the support as in the end where the roof appears like a succession of arches of 50 cm depth.

These characteristics remain in the final solution, though the radius of the successive transverse sections becomes variable, rather than constant, in the built form. For it, we can think that the structural variations were not made due to formal questions, since the definitive aspect is very similar. The reason for these changes was to simplify the roof construction, by proposing an easier and therefore cheaper construction method.

As stated earlier, the grandstand roof consists of a series of successive spans, mutually balanced, except in the ends, where vault sections work as beams in the horizontal plane. This way, the whole roof works like a complete structure, of which any intermediate element cannot be eliminated, all are needed to assure the stability. This arrangement led to a construction difficulty, since it was necessary to finish the construction of the whole structure, including the extreme buttresses, for a stable configuration. Therefore, it was necessary to maintain the formwork of the floor and the roof (around 5 000 m^2), until all the parts reach the necessary resistance. The result was a higher cost of formwork and scaffolding expense. The position of the ribs of the vault reinforcement, and the beams meant the roof was constructed in three phases, carrying out the slab first, next the reinforcing for the ribs and, lastly, the beams. Again, this resulted in an increase in formwork and construction time. Finally, the roof design, with the ribs and the beams on the exterior of the roof, hindered the water drainage.

In summary, the competition proposal has, at least, three difficulties:

a) It is necessary to maintain the formwork of the whole roof surface and the floor during the entire construction.
b) Difficulty of pouring concrete for the ribs and superior beams made it necessary for several successive phases.
c) A faulty solution to the water drainage.

THE MODIFICATIONS OF THE ROOF STRUCTURE

When facing the necessity of building the tribunes the difficulties of the initial proposal became obvious. Although the process of modification of the project, from the proposal of the competition until the definitive one, is not documented, the date of the project documents allow us to suppose that the new proposal was studied, after the verdict of the competition, between January and May of 1935. During this phase, Torroja carried out a change in the roof structure to solve the inconveniences.

The initial proposal for the competition could be considered like a conventional structure. For it, the ribs of the upper surface of the vault could be cast between the beams. Then the continuous beams would be supported on the cantilevers, and the curved shell built between the beams. Since the distance

between ribs is 2.45 m, it could have been solved with the 6 cm depth proposed. With this alternative the horizontal thrusts would be eliminated, and the necessity of constructing the whole roof once and for all, solving one of the difficulties, although not the other two.

However, from the outset of the competition, the authors show their desire to solve the construction using a shell structure. And, on the other hand, in the months lapsed between the delivery of the project, October of 1934, and the verdict of the competition, December of the same year, Torroja studied several projects of shells structures, some of those were built and they rehearsed several in the months previous to the delivery of the definitive project. In them he checked the possibility to span with concrete shells 5 cm thickness, and without the necessity of using reinforcement ribs in the upper face. On the other hand, the inconvenience caused by the excessive formwork expense in the shell structures was already evident at that time and proposals to solve it, like the marquees of the station of Munich and the garage of Nuremberg.

Torroja's role was decisive in this process, since his office defined the design of the roof. However, it is easy to think that the changes were not immediate, but the result of a series of rough calculations. For example, one of the drawings of the built project, plan nº 246.229 of the Eduardo Torroja Archive, dated May 1935 (figure 6), defines the geometry and the steel reinforcing of the transverse frame. A section corresponding to the roof element is also drawn whose ends are sections given by a vertical plane. Over that contour there is drawn another profile that cuts the element by an inclined plane. In this way, the crown of the arch at the edge of the shell extends further than the springing line, which is the final form of the shell.

As pointed out previously, one of the causes of the construction difficulties derives from the fact that each roof spans precisely the construction of the other ones to be stable. For that reason, the fundamental change that Torroja introduced was to substitute the vaults supported on beams, for independent cantilevers on each couple of supports, tied together, that can be built independently. Each section becomes a cantilever constituted by two sections of a surface of double curvature whose transverse section has the form of two circumference arches. The radius of these arches varies in each transverse section. The thickness of the resulting element is also variable, from 6 cm at the edge to 75 cm at the support. In the original option, two structures are combined: the cantilever beams and the vaults. In the revised and

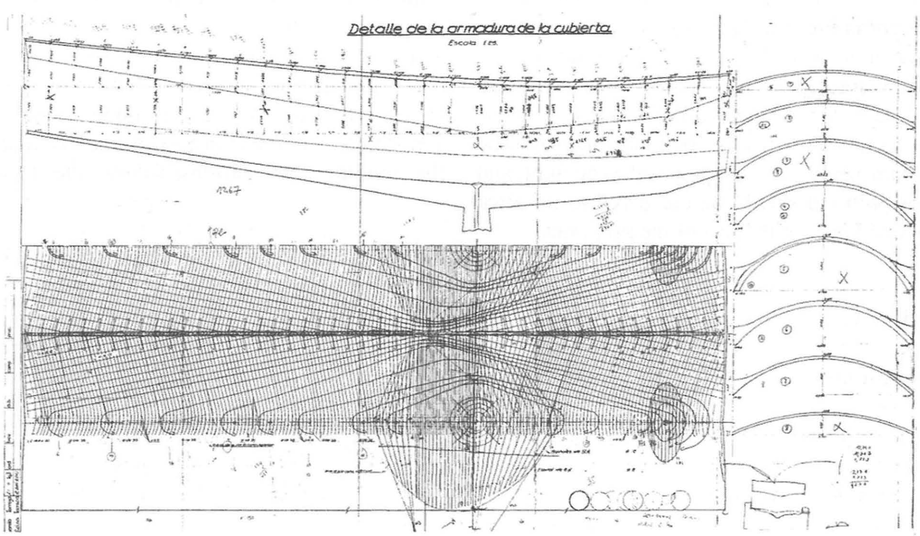

Figure 6
Geometry of the transverse frame

final version, the vaults are the only element, which form the roof and work as a cantilever.

In all the projects presented to the competition, the cantilever had a variable section, with a maximum depth at the support. Therefore, the edge of the roof element at the support will seem like that of the beam that solved the previous structure (in the case of the proposal of Torroja it was 1.50 m deep). On the other hand, one of the conditions of the project consisted on maintaining the form of the front of the roof as a succession of arches 50 cm deep. The roof should be, therefore, a continuous surface 50 cm deep at the border and 150 cm deep at the supports. From there several solutions were possible, such as using a straight line to support each arch, or any other curve type, to form a surface of revolution.

The hyperboloid of revolution is a ruled surface that Torroja had used previously, with the great advantage that straight reinforcing bars can be used which easily correspond to the ruled surface. Also, the surface of revolution can be formed using circular guidelines, whose laying out is easy. With this new structure Torroja solved the previous difficulties, since each element can be build independently of the rest and it is formed by a continuous surface of concrete without projections in their upper face. The concrete pouring can be done much easier as well. This way, the superior part of the roof is a continuous surface for water drainage. It is possible this way to solve the difficulties presented by the initial version.

Torroja modified the project for construction reasons, and the solution he finally adopted shows his ability to approach the problems, making use of the available technology, as a true building engineer. The work of Torroja allowed the proposal to be built, and the initial proposal should be compared to the final solution, and not with the rest of the proposals.

THE DEFINITIVE PROPOSAL

The grandstand

The section of figure 6 shows the transverse frame as it was built, which can be compared to the initial proposal of figure 3. Regarding the one proposed in the competition, the outer supports 1 and 4 of the lower floor were removed and the roof structure was changed. Support 1 could be removed without problems, because the span is equal to the thickness of the beam at support 2. The support 4 could be removed by taking advantage of the presence of the roof tie whose vertical reaction balances, partly, the weight of the terrace floor.

In the lower floor the same initial solution remained, using a shell structure of double curvature, with the section of a torus, but the stiffening ribs in the upper face of this surface were eliminated. When removing these ribs a new element appears in the section that is not mentioned in the proposal of the competition. It is the beam that unites the central supports in the tribune whose function is to stiffen the frames in the longitudinal sense. When eliminating the upper ribs of the roof, the shell is not, in the authors' opinion, sufficiently rigid to guarantee the stability.

The roof

The new structure of the roof is the most outstanding element in the design. The definition of the module was finished in May 1935 and, to explain it, a longitudinal section was drawn. This defined the dimensions at 28 transverse sections, situated every 75 cm along the cantilever, and at a variable distance around the 60 cm in the rest (figure 7).

The definition of the surface was carried out by thinking of its construction, since each section corresponds with an arch brace of the formwork on which is placed. Each transverse section consists of two circumference arches, which allows a simple laying out. However, the resulting figure is not a surface of revolution. In fact, in the extreme part of the module the sections follow the form of a

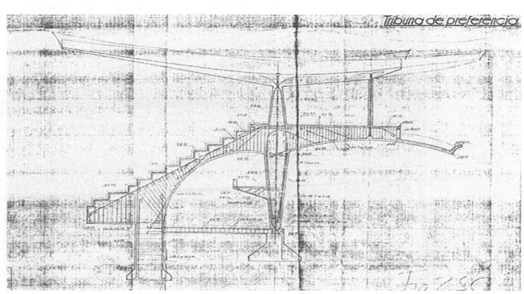

Figure 7
Geometry and reinforcing bars of the roof

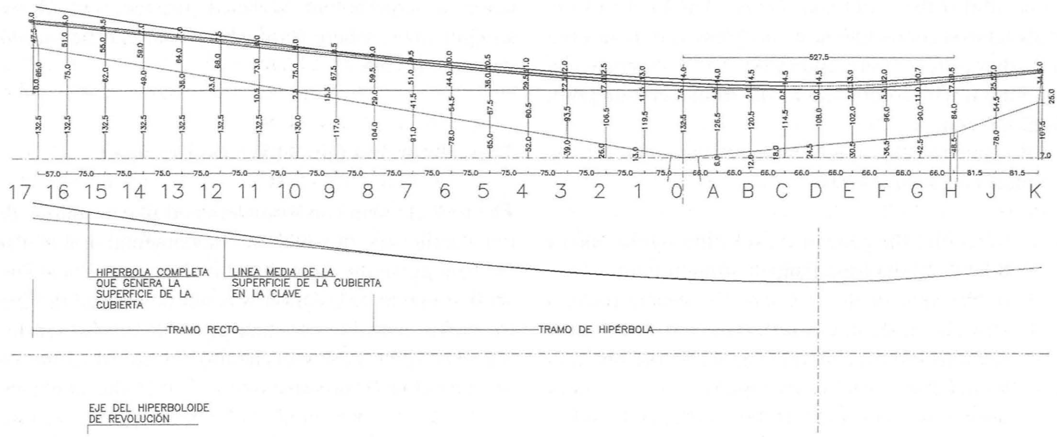

Figure 8
Dimension of transverse section

circumferential arch, but its radius is bigger than that of the corresponding surface of revolution.

With the elevations indicated in the section of the figure 8 each circumferential arch combines to form the lower face of the shell. With the bench marks that define the lower face of the section of the figure, the expression of the equation of a hyperbola has been deduced. This is adjusted quite well to those coordinates, approximately until the section nº 10, starting from which the layout coincides perfectly with a straight line. This way, the curve is defined as a hyperbola and a tangent straight line.

As has already been mentioned, the interior of the roof is not an accurate surface of revolution, because the figure that would be obtained doesn't coincide exactly with the built surface. In figure 9, the surface of revolution starting from the generatrix at the crown (with a horizontal axis) has been superimposed on the form as built. This last would give an elevation of the grandstand so that the roof edge would be a succession of arches of 70 cm depth, more than the arches of the initial proposal.

To obtain the edge for the shell, it was enough to increase the radius of the final arch until the necessary

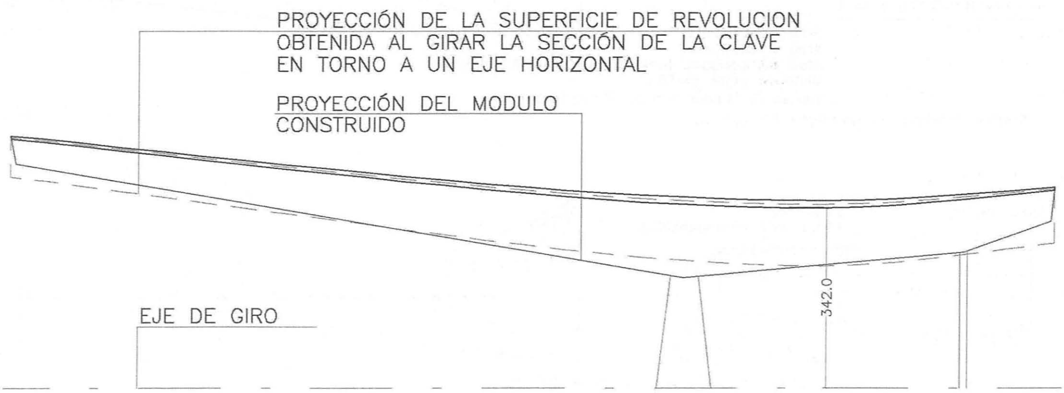

Figure 9
Module built with the surface of revolution superimposed

value so that the depth is achieved. The radius of the transverse sections increases progressively regarding the theoretical corresponding to a surface of revolution with a horizontal axis, from a certain point until the end.

In summary, the modification of the figure of the revolution has three phases:

a) to modify the generatrix, substituting the end of the hyperbola for a tangent straight line;
b) in the area in the one that the generatriz is a straight line, the radius of the transverse sections is progressively bigger to the one that would correspond to the figure of revolution of horizontal axis, until arriving to the end; and
c) the curve that is formed in the intersection of two hyperboloids is substituted by a straight line that wraps this arch (figure 9).

With these alterations of the theoretical surface the definitive form of the roof is obtained that adapts to the geometry of the initial solution and, at the same time, provides a way to generate the surface with the help of circumferential arches, allowing the construction advantages of a surface of revolution. On the other hand, in the area where most of the main reinforcing bars are located, the surface continues being a hyperboloid, a ruled surface formed by straight lines, where reinforcing bars can be placed without curving.

The reinforcing bars of the roof element

The roof element can be understood like a beam with two cantilevers, in which the fundamental problem is the bending at the support to the main cantilever. The shell structure calculation was not developed in time to make possible an analysis of the structure; however, there were precedents of lineal structures of unrectangular transverse section, like the aqueduct of Tardienta projected by Alfonso Rock whose transverse section resembles a circumferential arch.

In this case, the structure is analyzed like a continuous beam, and the calculation of the sections is made by graphic methods, like those proposed by Zafra. Applying this procedure to several sections of the roof module for the estimates of the self weight, they obtained the depth of the neutral axis and the moment of inertia of the section (figure 10).

The height of the solid part of concrete in the half area of the section is such that the neutral axis is always inside, so that the concrete of the shell is never compressed.

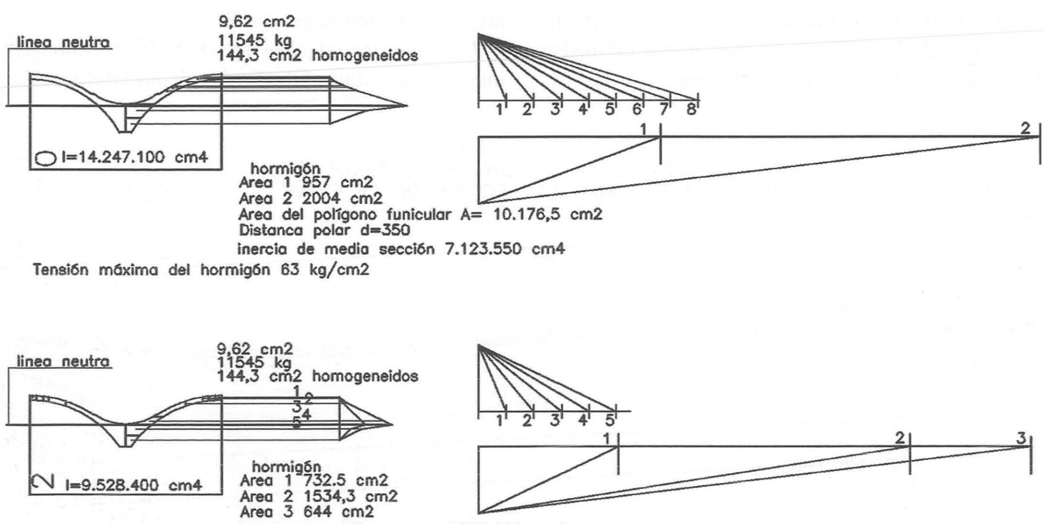

Figure 10
Graphic calculation of the section

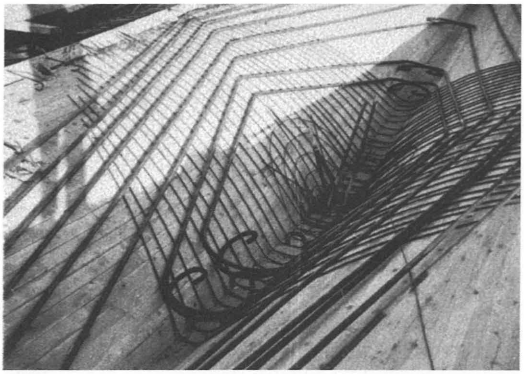

Figure 11
Reinforcing bars in the module

The reinforcing bars organization

To organize the reinforcing bars of the module they used the current methods that consist in adapting the bars to the isostatic stresses. Therefore, the analysis of the structure aimed to obtain the form of these curves. In accordance with this approach, the reinforcing bars were prepared as shown in figure 11.

This form of distributing the bars has a difficult point, in the area where the main bars bend at the line of the supports. At this point, the radial component of the bent bars compresses the concrete of the shell in the transverse sense. To avoid tension at that point they dispensed with the bent bars and increased the thickness of the shell in that edge.

The rehearsal of the module and the construction

The company in charge of the construction of the race track, Agroman E. C. S. A., was carrying out several works in the Ciudad Universitaria de Madrid at the time, including the building of the *Facultad de Ciencias*. Its director, Agustín Aguirre, offered the possibility to study a module at full scale.

With this initiative it was gotten, besides the structural load test, the opportunity to study the constructive process, checking the viability of the disposition of the reinforcing bars. In fact, the plans that define the structure are of May 1935, as has been indicated. However, the plan defining the geometry of the roof and the dimensions and placement of the reinforcing bars, plan 246.228,1, is of June 21, more than one month later. Also, the notation 228,1 was usually used in the Technical Office to designate a plan that substitutes another, which would be the 246.228 in this case. It is probable, then, that they made an initial documentation to carry out the module of the load test and, when making it, adjust the dimensions, including the arch braces and formwork, as well as the length and bent of bars. The final result would be reflected in the plan made later, which is the one that is conserved.

In the rehearsal, the test module was loaded until failure, which occurred with a total load of 605 kg/m^2. During the course of the loading they registered the efforts taken place in the compressed area and the deformation in the ends of the cantilever that, in the inferior vertex, arrived at 15 cm, and something more in the lateral ends of the edge, since they were not supported with other adjacent sections. It was observed that the transverse deformation in the area next to the support was small. The images of the broken module show two aspects of the behavior of the shell. In the first place, the main work is the bending of the cantilever and the cause of the failure were the radial compression of the main reinforcing bars in the area next to the support were they are bent. (figures 12 and 13) On the other hand, the test illustrated the importance of the deformation of the lateral ends of the shell, regarding the central vertex.

Once the pattern and the load test were carried out, construction began and, in July of 1936, it was

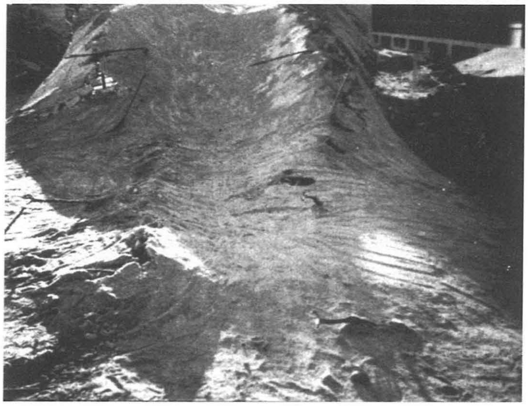

Figure 12
Frontal view of the broken model

Figure 13
The broken model

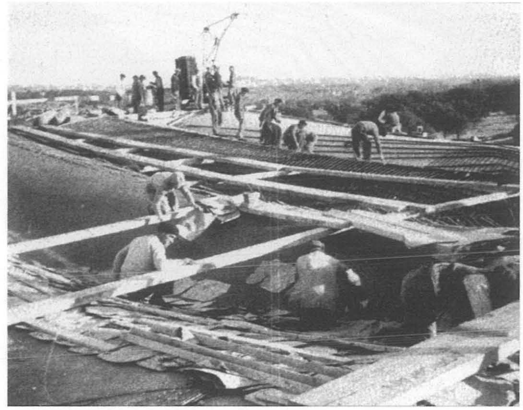

Figure 14
Phases of the roof construction

practically finished. To carry out the formwork of the roof several modules were used successively, so that the necessary scaffolding it occupied only a part of the total longitude of the roof. With that organization, the concrete was poured for modules shown in figure 14, where a completed section appears while the next section is poured, leaving the disposition of the reinforcing bars in the following one and the beginning of the assembly of the last section.

8. Damages during the Spanish Civil War. Repairs and inauguration

During the Spanish Civil War, the area was a battle front and the buildings received numerous bombing impacts. This produced several perforations in the roofs, many of which exposed the reinforcing bars. Although none of the roof sections collapsed, the roof suffered many fractures, as a consequence of the oscillations caused by the explosive waves of the impacts. The perforations in the sheet were repaired using a formwork with boards of the same dimension as the original, though their localization was difficult. The ends of the cantilever of the final modules only needed to be rebuilt. As it was observed in the load test, these ends had been deformed more than the central vertex of the cantilever, due to the lack of adjacent modules to provide support. This was the situation of the final modules of each grandstand where they did not have an adjacent support, causing excessive deformation. To solve it, these ends were rebuilt again, and reinforced with five diagonal ribs located in the upper face of the shell.

Once the damages were repaired and the ends of the roofs rebuilt, the complete surface was waterproofed, something that had not been carried out before the war. With these modifications, the works of the Zarzuela hippodrome ended in time for the inauguration in May of 1941.

Notes

1. Antuña Bernardo, Joaquín. 2002. The structures of construction of Eduardo Torroja Miret. Doctoral thesis, ETSAM, Madrid. (advised by Ricardo Aroca Hernández-Ros). This thesis contains the complete bibliographical information.

Structural characteristics of the elliptical masonry dome of the sanctuary of Vicoforte

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Mario Alberto Chiorino
Roberto Roccati

The elliptical masonry dome of the Sanctuary of Vicoforte near Mondovì in northwest Italy, built in 1731, is the largest dome of this kind in the world (Figures 1–4). With its exceptional dimensions (major and minor internal axes 37,15 and 24,80 m, maximum height of the monument 84 m) it largely surpasses the dome of S. Andrea del Quirinale (1658) by Bernini (whose dimensions are 25,80, 16,25 and 25,80 m, respectively) and a group of other elliptical domes both in Rome (S. Carlo by Borromini, 1638, and S. Giacomo by Volterra, 1592) and in Spain (S. Hermenegildo in Córdoba, 1616 and Convento de las Bernardas in Alcalá, 1626) with major axes between 23 and 25 m and minor axes between 16 and 19 m (Zocca 1945, Escrig, Cobreros and Valcárcel 1997). Larger dimensions can be found only in the small group of famous axi-symmetrical domes comprising the circular domes of the Pantheon (2nd century, internal diameter 43,30 m) and San Pietro in Rome (16th century, 42,84 m), and the octagonal dome of Santa Maria del Fiore in Florence (15th century, 42,0 m diameter of the inscribed inner circle). The circular domes of Hagia Sophia in Istanbul (6th century) and of St. Paul in London (17th century) have an inside diameter of about 33m.

These exceptional dimensions are to be related to the official role originally conceived for (but never assumed by) the monument intended to become the mausoleum of the Savoia family that governed Piedmont and southeast France since 15th century.

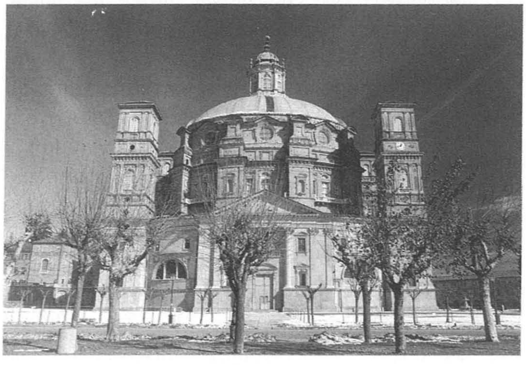

Figure 1
General view of the Sanctuary of Vicoforte (Mondovì, Italy)

Figure 2
External view of the dome

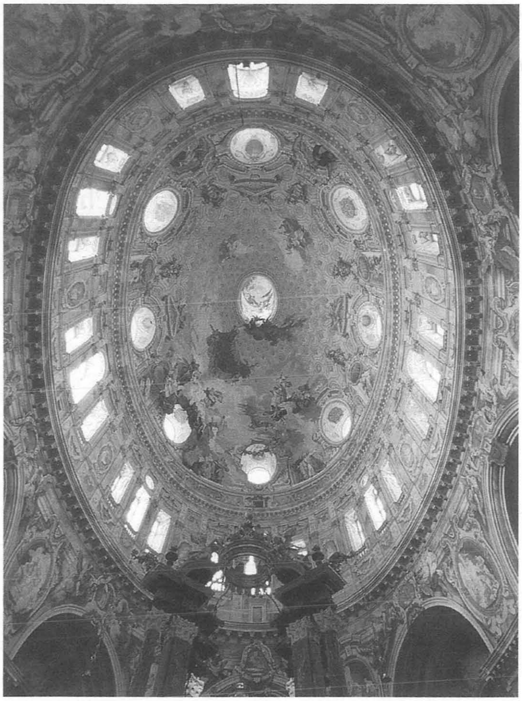

Figure 3
Internal view of the dome

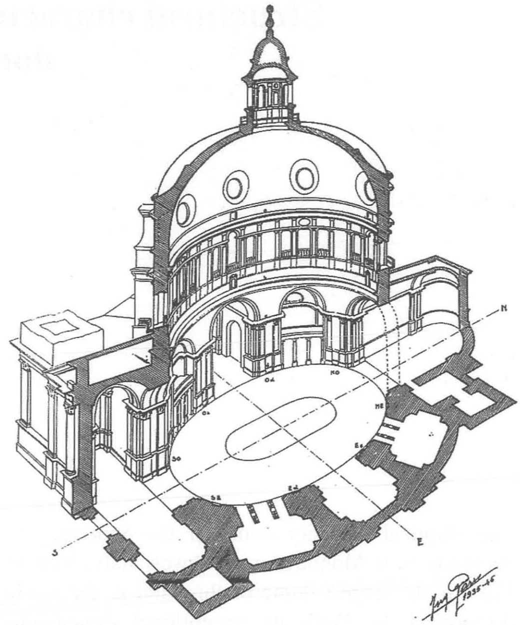

Figure 4
Structure of the Sanctuary of Vicoforte (Garro 1962)

Unfortunately the stability of large masonry monuments, and of big domes in particular, is threatened by progressive fracture due to aging and chemical degradation of materials combined with the static and dynamic effects of dead load and ambient actions (thermal loads, wind, seism, traffic, etc.), settlement of foundations, yielding or delayed rupture of original iron reinforcements, etc. (Como 1998). The exceptional dimensions of the Sanctuary of Vicoforte and especially of its dome, together with the peculiar geometrical shape of the dome itself and the particular slenderness of the drum-dome system (which are responsible of a complex and particularly severe static behaviour), and, last but not least, the unfortunate selection of the site of the Sanctuary from a geotechnical point of view, have largely increased the fragility of this monument, which has suffered significant structural damages during more than two centuries and was consequently submitted to an important intervention of structural strengthening two decades ago.

The principal structural damages consisted in:

— very large foundation settlements during the construction of the massive lower parts of the monument at the end of 16th century, and after the construction of the dome in the 18th century (maximum differential settlements of the west-side foundations with respect to the north-east side, developed during the whole history of the monument, were estimated in 1935–45 to be of the order of 33 cm) (Figure 5, Garro 1962),
— progressive opening of large meridional cracks reaching a maximum width at the base of the dome of 82 mm for a single crack, and a total amplitude along the perimeter of 413 mm (Figure 6, Garro 1962).

The project relating to monitoring, rehabilitation and structural strengthening was started in 1976 and (a) structural and geotechnical investigations of foundations and foundations layers, (b) rehabilitation of the drainage system of clay-silt layers, (c)

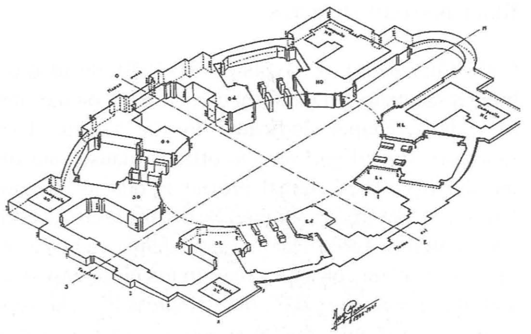

Figure 5
Axonometric view of foundation settlements (Garro 1962)

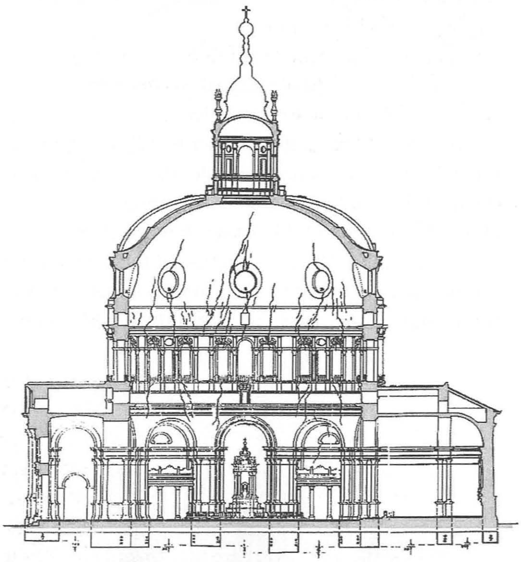

Figure 6
Cracks of the dome. North-south section; view of west side (Garro 1962)

structural strengthening of the dome through the formation of a post-tensioning ring at the base of the dome, and (d) monitoring of the principal parameters characterizing the structural disorder and of the response of the monument after strengthening were carried out (Pizzetti and Fea 1988, Chiorino, Fea and Losana 1993).

A new five years program of monitoring, research and investigations —named *Vicoforte 2002–2006*— aimed to the control of the present conditions of structural conservation of the monument and to the establishment of the correct criteria for its future maintenance and, where needed, further structural restoration, was started in January 2002 under the coordination of the Politecnico di Torino and the responsibility of the second and third authors. As a part of this program a joint project of research has been established between the Politecnico di Torino and a group of Japanese Universities and research institutions under the coordination of the first author.

The objectives of the program *Vicoforte 2002–2006* are the following:

1. Improvement of the geometrical and structural description of the monument
2. Diagnostic inspection of deterioration and structural damage by means of non-destructive tests
3. Characterization of the properties of the masonry by non-destructive and, where feasible, partially destructive tests
4. Investigation of the mechanical characteristics and continuity of the three original sets of annular iron ties embedded at the base of the dome
5. Geotechnical characterization of the foundation layers by field and laboratory tests
6. Installation of a new and extended monitoring system for the continuous control of the main parameters characterizing the behaviour of the monument, with special regard to the dome, and of the foundation layers
7. Measurements of natural frequencies and modes by ambient vibration and continuous registration of the dynamic responses to seismical events of selected magnitude
8. Interpretation of the static and dynamic behaviour of the dome and of the monument as a whole by means of finite element three-dimensional elasto-plastic and dynamic analyses, including its foundations layers, with inclusion of proper damage criteria
9. Updating of numerical models by comparison with experimental measures
10. Estimation of the global safety of the drum-dome system by limit analysis
11. Proposals for structural conservation and maintenance with special attention to the post-

tensioning ring at the base of the dome and to the definition of optimum stress levels for the steel tie-rods

12. Analysis of the site of the monument from the hydrological point of view and estimation of the risks connected with extreme events.

The Italian team of research addresses in particular objectives 1, 5, 6, 10 and 12, while the Japanese team focuses mainly on objectives 2, 3 and 4, the remaining objectives are addressed as a joint project.

Preliminary results concerning (a) diagnostic, inspection and material characterization of the monument, and (b) estimation of global safety of the dome-drum system by limit analysis will be published in separate papers. The present paper addresses essentially point 8 and presents the preliminary stages and results of the research program concerning the numerical modelling of the entire monument.

A series of partial analyses by finite element method (FEM) limited to the dome alone were performed in the past years in connection with the rehabilitation works (Bernasconi and Marchini 1979), and subsequently in the frame of a research program at the Politecnico di Torino (Cecca 1994). The present work presents for the first time a FEM three-dimensional analysis of the entire construction (Yamaoka 2002). At this stage of the research program the mathematical model does not yet include the foundation layers, whose geotechnical characterization will result from step 5 above, and the structural characteristics of the masonry construction are investigated on the basis of linear-elastic analysis. Inclusion of foundation layers and consideration of damage criteria and influence of crack formation and propagation will be part of a further study.

By consequence the present study must be considered only as an initial exploration of the structural geometry and of the basic static pattern of the monument. Nevertheless, some preliminary indications on the critical aspects of its structural configuration can be obtained and they can help both in the definition of the further steps to be accomplished for the refinement of the numerical model, and in drawing the general lines for the program of monitoring and of experimental tests.

Brief historical notes

Carlo Emanuele I of Savoia (1562–1630) decided to build a sanctuary, in the form of a basilica, on the site of a holy chapel dedicated to the virgin. The sanctuary should become the official mausoleum of the dynasty. The original project is due to Acanio Vitozzi (1539–1615) (Figure 7) and the construction was started in 1596. Due to an inadequate choice of the site (1/3rd on consistent marl in the north-east side and the remaining 2/3rd on compressible clay-silt layers of variable thickness up to 3,5 m) large differential settlements took place and the construction was practically abandoned at an elevation of 10 m at the end of 16th century, while continuing drainage works in the clay layers during the first part of 17th century.

In 1715 Vittorio Amedeo II di Savoia (1666–1732) decided to build the Sanctuary of Superga on the hills near Turin as the new mausoleum for the Savoia dynasty, and assigned the project to Filippo Juvarra. Although the representative role of Vicoforte was consequently fundamentally lost, architect Franceco Gallo (1672–1750) of Mondovì convinced the royal family to complete the construction of the basilica and built the baroque dome in 1731 after compensation of settlements at the base of the drum. Differential

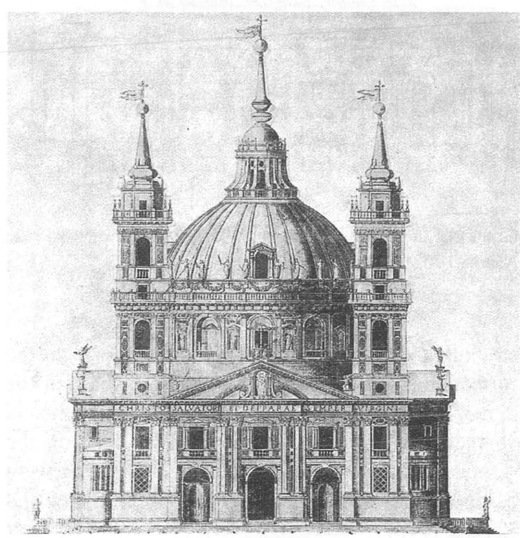

Figure 7
Original project by Ascanio Vitozzi (1596)

settlements developed again due to the new weight of the dome, and cracking of the dome and the drum started developing and progressively increased until the recent application of the strengthening system in 1985–87. No substantial increases in the crack widths were observed afterwards.

Geometry of the dome

The horizontal section of the dome, normally referred to as elliptical, appears to be, after inspection of the photogrammetrical survey of the intrados of the dome, an oval consisting of 4 circular arches, very close to an ellipse. The curvatures vary moving from the base of the dome to the lantern, due to the different ratios of the diameters. The base internal diameters are 37,15 and 24,80 m (ratio 1,5) and the diameters at the base of the lantern are 7,74 and 5,94 m (ratio 1,3) (Figure 8).

The internal height from the base of the dome to the lantern is 16,66 m. The internal meridional curves that can be obtained from the photogrammetrical survey can best be approximated by arches of ellipses. It can be presumed that the control of the geometry during the construction was performed on the oval parallels, easy to be traced.

The extrados curve of such a baroque dome is more complex. The thickness of the dome varies from 1,91 m (1,70 m) above the windows to 1,37 m (1,17 m) in the proximity of the lantern for the transversal (longitudinal) cross sections (Figure 9).

Original circular ties and recent strengthening

The dome and the drum were originally strengthened by circular iron ties at three different levels, with a total cross section of about 140 cm^2 (Figure 10). A

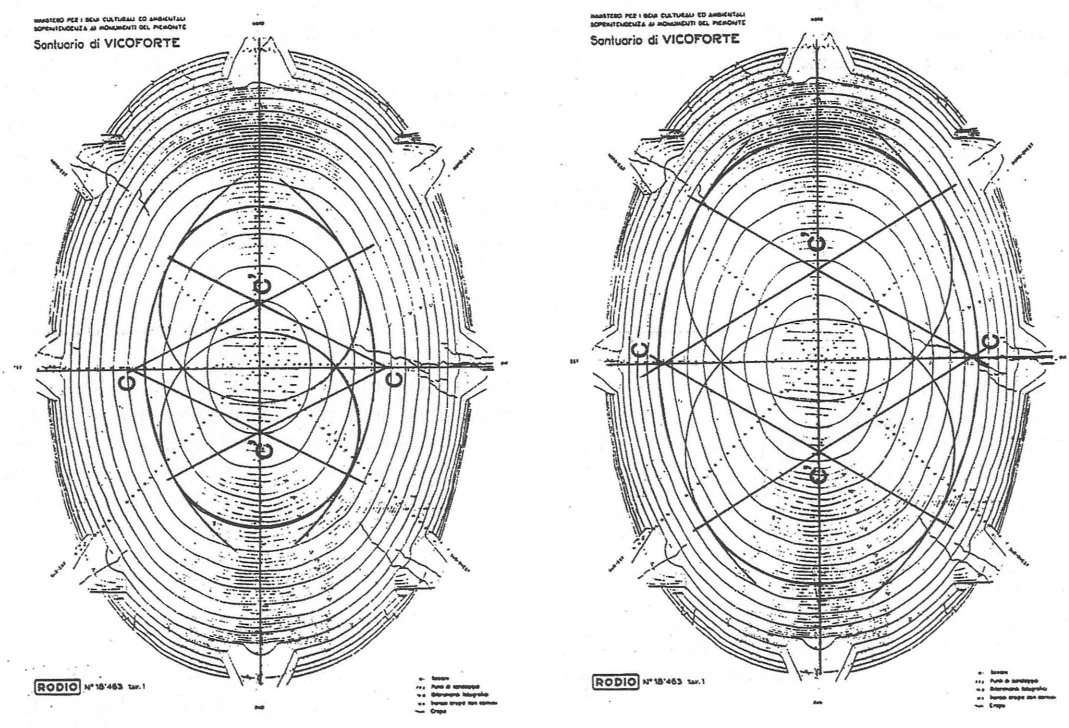

Figure 8
Photogrammetrical survey of the intrados of the dome. The parallels are approximated with ovals consisting of 4 circular arches

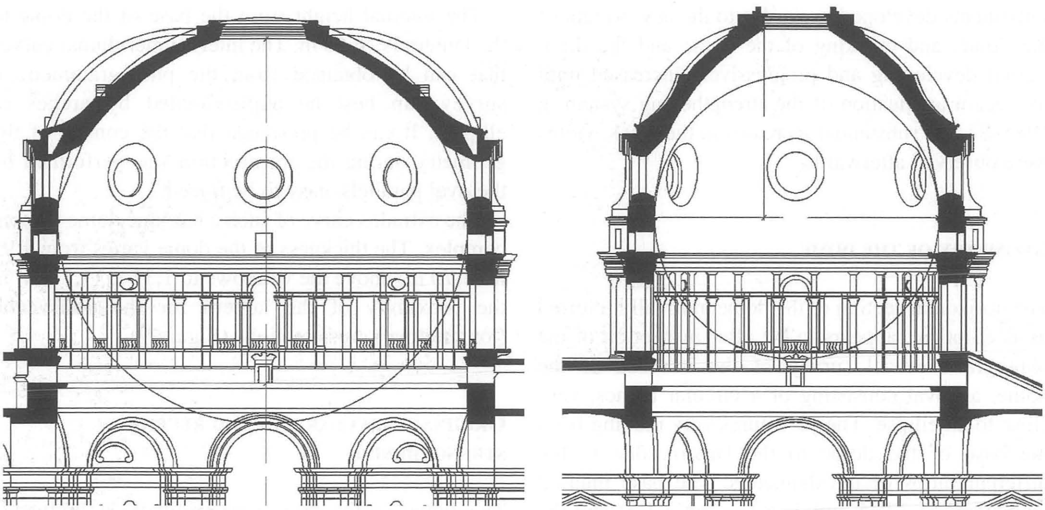

Figure 9
Cross sections of the monument along the main axes

program of non-destructive tests is presently under way to ascertain the mechanical integrity and characteristics of the original tie-system.

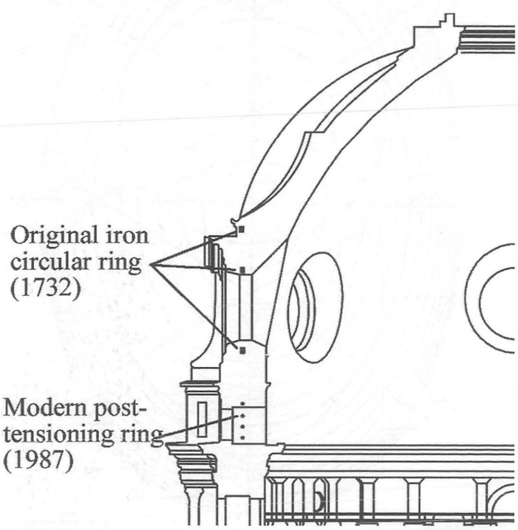

Figure 10
Cross section of the dome with indication of original iron circular rings and modern post-tensioning ring at the base of the drum

It may be interesting to observe that the cross section of the original ties for the dome of San Pietro inserted in 1588–90 by Della Porta at the moment of the construction (1588–90) reached only 60–70 cm^2, while the effective section of the additional strengthening rings applied by Poleni and Vanvitelli at the moment of the structural rehabilitation of the same dome (1743–48) reached 207 cm^2.

The modern strengthening system consists of 14 groups of tangential ties, each group consisting of 4 superimposed Dywidag 32 mm bars (32 cm^2 in total) of high-strength steel, normally used in prestressed concrete constructions, each bar being located in ducts drilled in the masonry (Figures 10 and 11). The 14 groups of bars are interconnected by steel trusses. The force in the tie-bars may be regulated at any time by jacks and the stress is constantly monitored by load cells.

Monitoring

The monitoring system includes, beside the load cells at the tie bars, a large number of displacement measuring devices (partly mechanical and partly electrical and automatic) applied to the main cracks, and temperature gauges. Measures of variation of

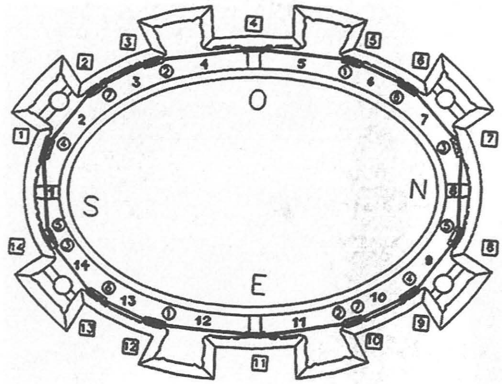

Figure 11
Post-tensioning ring at the base of the dome

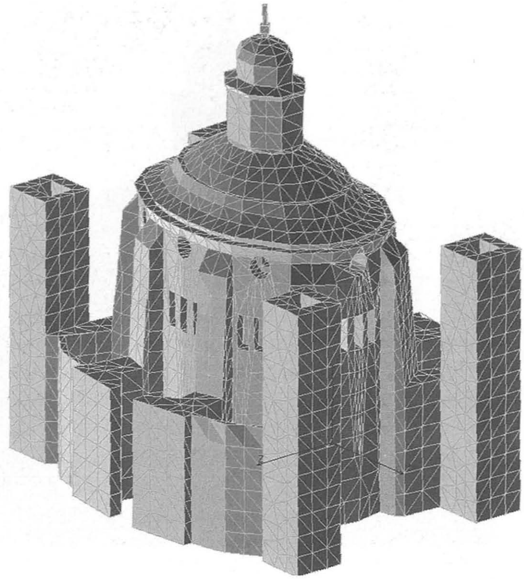

Figure12
Numerical model

diameters have been performed manually until now. At present the monitoring system is under complete renovation and will include appropriate instruments for automatic measure of dimensional variations, crack openings, differential settlements, and accelerometers for measuring the response to natural (seism and wind) or artificially induced vibrations.

Numerical model

As a first stage of analysis, the structural characteristics of the monument are investigated on the basis of linear-elastic analysis. The model, which is composed of 10-node tetrahedral solid elements, is shown in Figure 12. A second model composed of 20-node hexahedral solid elements is presently under construction and will be used for the second phase of the analysis in the non-linear domain. Cosmos 2.5 is used in this analysis. Total number of 10-node tetrahedral solid elements and nodes are 32.181 and 16.716, respectively. Based on experiments (Barosso 1979, Rodio SpA 1983), material constants used in the finite element analysis are determined as follows: Young's modulus 15.000 kgf/cm^2, Poisson's ratio 0,15, weight per unit volume of the masonry 1.700 kgf/m^3, compressive strength 30 kgf/cm^2, tensile strength 3 kgf/cm^2.

Preliminary Results of the analysis

The result of the analysis in case of dead load is shown graphically in Figures 13 to 15 representing the state of deformation. The ellipse delineated by the

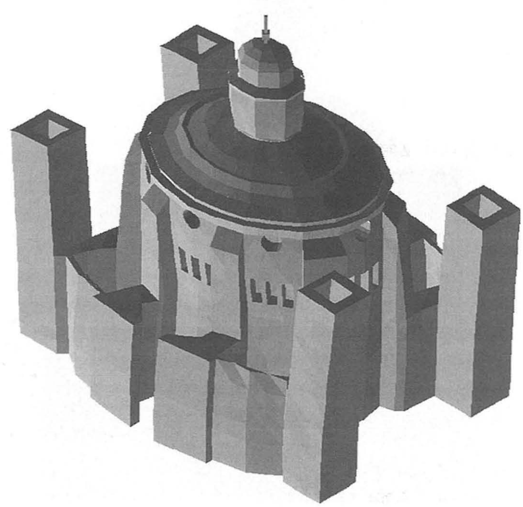

Figure 13
Deformation

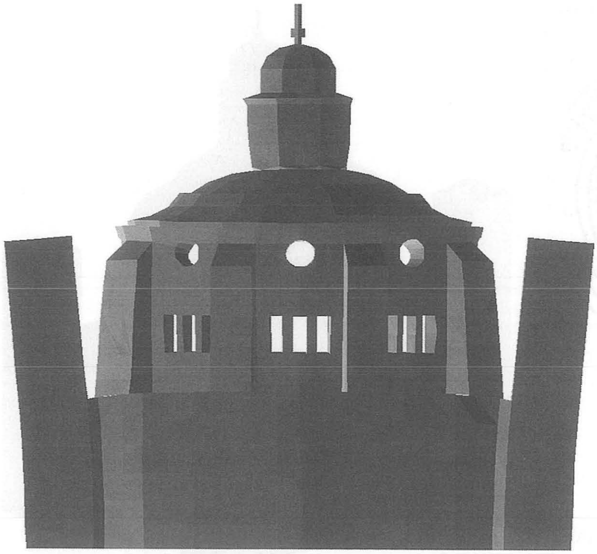

Figure 14
Deformation in north-south direction

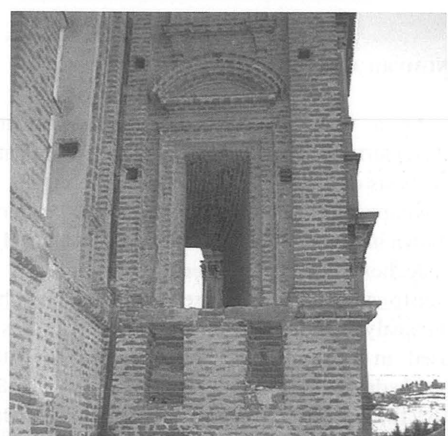

Figure 15
Deformation in east-west direction

base of the dome deforms into a more acute-angled ellipse. The massive buttress piers on the north and the south sides deflect outward due to the thrust of the dome and diagonal crack develop, as can be seen in Figure 14. On the eastern and the western sides, however, as shown in Figure 15, the upper parts of the massive piers deflect inward due to the eccentricity of the given load against the lower supporting structure. This oval displacement pattern of the base of the dome is in accordance with the actual state of the structure.

Principal stress 1 is shown as a vector field in Figure 16. The vectors with dark colour represent large tensile stresses. The crack of masonry is assumed to occur when the tensile principal stress exceeds the tensile ultimate strength. The direction of the crack is perpendicular to the direction of the tensile principal stress. According to the direction of the principal tensile stress, the vertical cracks of the dome and drum correspond well to the computed state of stress of the whole structure (Figures 6 and 17).

Concluding remarks

Though the actual deformation is too large to be analysed in the elastic phase of the material, our results correspond well to the oval deformation observed along the base of the dome. They show also that the locations of both the original annular iron ties, of inadequate section, and of the post-tensioning belt embedded at the base of the dome in 1987 were properly selected.

The results of non-destructive tests will be used for updating the numerical model. A second phase of the study will concern elasto-plastic analysis with inclusion of proper damage criteria for the material.

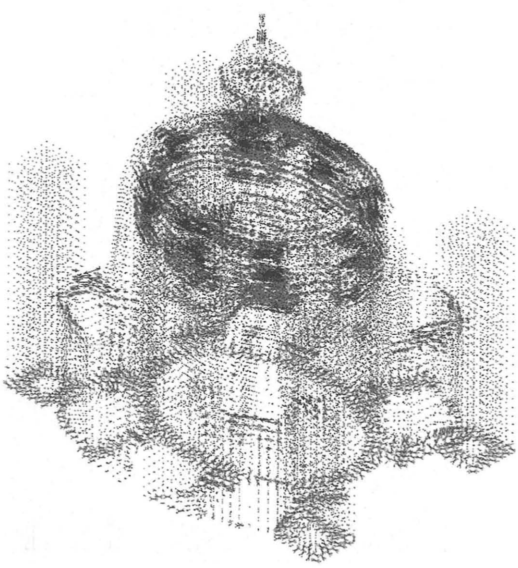

Figure 16
Principle stress 1

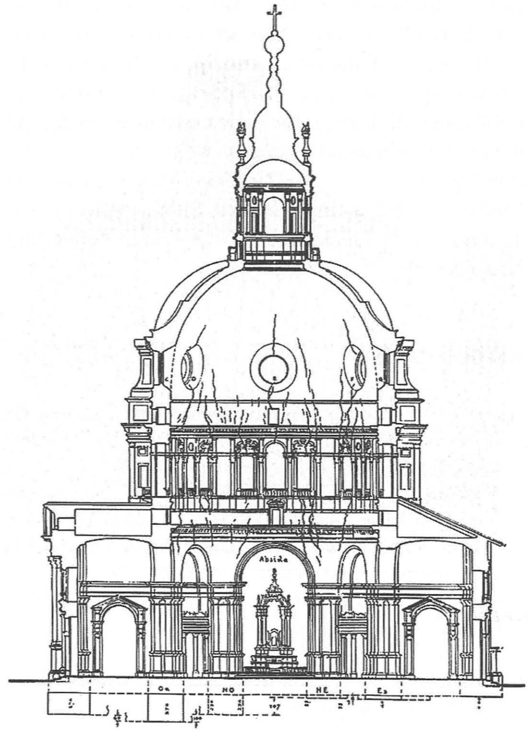

Figure 17
Cracks of the dome

Acknowledgements

This work is a part of the surveying project *Vicoforte 2002–2006* of the Sanctuary of Vicoforte which has been conducted from 2002 as a joint research between Italian and Japanese scholars. The authors wish to express their gratitude for the generous understanding

and cooperation of the Administration of the Sanctuary of Vicoforte. Appreciation is also extended to Hirokazu Yamaoka, student of Nagoya City University and Alberto Spadafora, student of Politecnico di Torino, for their assistance during the modelling. Financial supports were offered by the Fondazione Cassa di Risparmio di Cuneo, the Ministry of Education, Culture, Sports, Science and Technology of Japan, The Hibi Research Grant, and Nagoya City.

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Early Islamic lime kilns from the Near East. The cases from Amman Citadel

Ignacio Arce

The aim of this paper is to discuss the lime production methods in the Near East during the Umayyad period, with special attention to the relationship between kiln design, production processes, local conditions (in terms of resources availability), and the characteristics of the final outcoming product.

The analysis includes a review of the tradition of lime production in the region from Roman to Medieval period. Several aspects will be analyzed in detail, among others the differences in kiln design according to its use, comparing several samples found at Amman Citadel, Jerash (Jordan); St Anne Church complex in Jerusalem, the Herodion palatine complex (Palestine), etc. This will permit to establish a sound basis for the future study of the diachronic evolution of production as well as the synchronous comparision of different kinds of kiln according to their use.

The research conducted is part of the campaigns of excavation and restoration of several Umayyad monuments in Jordan carried out by the Spanish Agency for International Cooperation (AECI) and directed by the author since 1995, namely, the Umayyad Medina and Palace at Amman Citadel and the Qasr Halabat-Hammam As-Saraj complex. This research includes an ongoing regional sampling campaign of mortars, plasters and renders, that covers the whole region and spans from Roman-Nabatean times till Islamic and Crusader period.

Umayyad Lime mortars. The problem of the mortar composition

During the Umayyad period the most usual lime mortar used is characterized by its greyish colour, due to the presence of big amounts of ashes. These ashes are from vegetal origin and their presence is rather odd as this kind of ashes are quite hygroscopic and tend to debilitate and decompose the mortar, instead of reinforcing it. The presence in the region of Roman lime mortars with *pozzolanic* ashes (for instance in the Roman North Temenos wall at Amman Citadel itself), would mislead to the hypothesis of a failing attempt to reproduce the Roman mortars during the Umayyad period, without knowing the precise sort of ashes required to be added to the mix, in order to achieve the desired results.

Nevertheless, this idea does not fit with the evidences: On the one hand we can find also in Umayyad period fine lime mortars mixed with other hydraulic additives: For instance, pure white lime mortar with «*cocciopesto*» (i.e. crushed bricks as hydraulic additive), that can be found in the Umayyad hydraulic system at Amman Citadel, in the floor of Amman Citadel and Halabat mosques, as well as in many other Umayyad sites (Arce 1999 —in press—; Arce 2000 —in press—). Furthermore, as part of the systematic sampling of mortars carried out at Amman Citadel and throughout all the Near East, new kinds of Umayyad lime-based hydraulic mortars have been

identified, as the one from the walls of the mosque at Halabat. In this case, a lime mortar mixed with volcanic ashes (that certainly provide the mix with hydraulic properties) was found. This, as well as the mentioned samples of lime with «*cocciopesto*» prove undoubtedly that a precise knowledge of hydraulic additives was available to the masons working during the Umayyad period. Besides, and also as a result of this regional sampling, it has been found out that this kind of greyish mortar with charcoal and vegetal ashes[1] was used also in monuments and structures from the Roman and Byzantine periods (for instance in the foundations of the Roman colonnated street at Biblos in Lebanon).

These evidences, specially the fact that this kind of mortar was not devised in Umayyad times but earlier (although its use would be widespread during this period), corroborate the survival of Roman-Byzantine building techniques and know-how during the Umayyad period in the region (something suggested by many other evidences, see Arce 2000 & Arce 2001). Thus, another *raison d'être* should be sought for this kind of mortar, as the presence of ashes in this characteristic «Umayyad» lime mortar would not be the result of a deliberated addition of a component with specific properties, but the result of certain production process that could not avoid several by-products in the lime itself. This became more clear when, during the excavation of the East street of the Umayyad Medina at Amman Citadel, a singular lime kiln was found.

The Umayyad Lime Kiln from the Umayyad Medina at Amman Citadel

Location

During the 1998 campaign, the remains of a lime kiln from Umayyad period were found at the northern end of the East street, close to the Roman Temple dedicated to Hercules.

It is interesting to clarify the reasons for the location of the kiln, as it could be regarded apparently, as an unusual place for building a lime kiln: The location was determined by the proximity to the «quarry» (actually, the Hercules Temple was used as a quarry providing pre-cut stone and marble for the lime production), and to the worksite itself (the palace and the palatine city). It must be pointed out that during the Byzantine and Islamic periods no structure was built over this Temple and its immediate surroundings due to the sacking and reuse of its building materials (the nearby Byzantine Church —6th.C.AD—, for instance, was built almost entirely with *spolia* from this Temple). Meanwhile, its location in the eastern side of the hill is most probably conditioned by the prevailing winds —coming from the West— so that the smoke would not annoy any existing building (at Jerash can be traced a similar pattern in the location of the Early Islamic lime and pottery kilns —personal comunication of Dr. Alan Walmsley during the 3rd International Conference on the Archaeology of the Near East. —3ICAANE— Paris Sorbonne, April 2002). These lime kilns from Jerash prove that the location of such a polluting infrastructure within the premises of a city, was not unusual in the region at that time. In Pompeii can be found an interesting example within the «house of the Iliaque chapel» (Adam 1984, 75). Also in Medieval western Europe, lime kilns are placed, when possible, in the worksite itself (usually within the city). For instance in 1347, during the construction of the Dome at Orvieto (Central Italy), a lime kiln was built «nel chiostro dell'Opera, al di là della piazza, successivamente destinata ad uso della bottega —pontica— posta *"ne la piaçça de la chiesia nel cantone de la strada che va de a Sancto Francesco ne la quale se fa la colla per lo musaço"*» (Riccetti 1988, p. 164). Otherwise, in most cases, the kiln is placed as close as possible to the quarry. In our case both conditions are given, as the remains of the Hercules Temple was the actual quarry for both the already-cut ashlars for building, and the marble for lime production: Few remains of a monumental statue of Hercules coming from the Temple (just a hand and a fragment of an elbow) survive. Most probably, the rest of the statue, toghether with the Temple marble veneer, were burnt in this kiln.

Description of the kiln

It was built with bricks (30 × 30 × 7,5–8 cms) within a cylindrical ditch excavated in the sloping ground of the hillock placed in the center of the Citadel. It has an inverted tronco-conical shape (1,60m in height; 3,10 m upper diameter; 2,10 m lower diameter), with a flat circular base. The top of the perimetral wall has

a smooth transition, opening and reaching the ground floor level (no traces of any structure built over the ground are ascertainable). At the top of the west side of the perimetral wall there is a built-in hooper or chute (Figs. 1–3), that tappers off down into the kiln (lenght: 90 cm; external width: 60 cm aprox.; internal width: 50 cm). At the base of the kiln there is no opening apart from what seems to be a small flue between two bricks (8 cm in diameter, most probably an air inlet). The kiln presents evidence of use even after the 749AD earthquake (i.e. during the Abbasid period) that destroyed the Citadel: It presents several refurbishments, specially in its eastern side, where missing bricks have been replaced by stones. It is clear that it has been in use for a long time, although in the lattest periods the floor was raised, and probably it was not used just for burning limestone, but also for melting metal, as some metal slag was found in the lattest layer of use (corresponding to the Fatimid period). Near to it, there are the remains of a cistern cut in the bedrock (most probably from Byzantine period) that could have been used as a slaking pit. This cistern was originally covered by a barrel vault of masonry that during the Umayyad period was not standing anymore (Arce1999 —in press—).

Figure 1
Amman Citadel. Umayyad lime kiln. General view

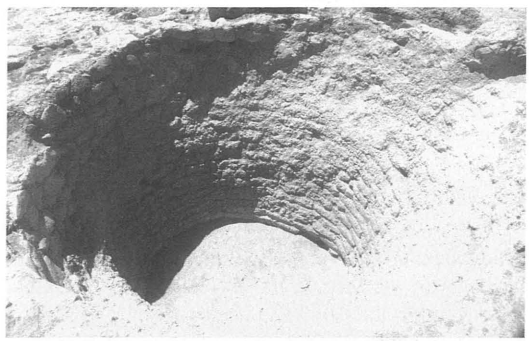


Figure 2
Amman Citadel. Umayyad lime kiln

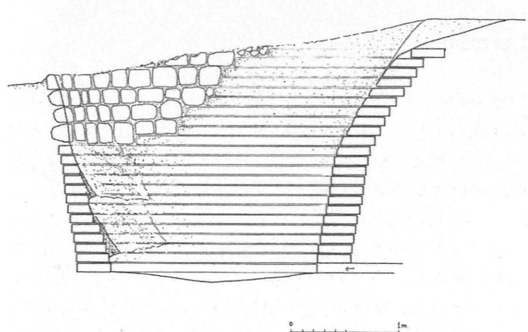


Figure 3
Amman Citadel. Umayyad lime kiln. Section

Operational hypothesis [2]

At first, the lack of a lower mouth or opening for feeding with fuel the burning chamber, raised doubts about the use of this structure (this led to think that it was not a lime kiln, but perhaps a slaking pit), but the traces of slag, ashes and lime, mixed with lumps of partially cooked lime and marble, found at the base of the structure confirmed its use as a kiln. Thus, apparently the operating process of this kiln was different from the usual ancient process of lime production known. Regarding the operation of this kiln, two hypothesis can be raised :

According to the first, this kiln would be a version of the traditional system («high flame» firing kiln), and consequently operated in a similar way: A sort of dome or pile would be built with the stones to be burnt on top of the furnace chamber, that would be feeded with fuel from the hooper. Once the limestone was fired, the pile of burnt stones (already transformed into quicklime) would be dismantled and removed to the

slaking pit. But in the standard kiln of this kind the opening for feeding the fire is placed at the bottom of the furnace, not at the top. This is the case, for instance, of the kiln from the Herodion fortress near Bethlehem dated in the Byzantine period, and the Crusader lime kiln from Saint Anne Church in Jerusalem (Fig. 4), both in Palestine. These two kilns also present at the top of the furnace chamber, the typical perimetral step required to built this sort of «dome» over it. But in our case this perimetral step does not exist, something that would stand against this hypothesis. At Amman Citadel, there is a second lime kiln, from Fatimid period (apparently kept in use till the Ayyubid-Mamluk period) with this «standard» design (mouth at the base of the furnace chamber and perimetral step at the top), and operational concept (see below), showing that it was well known in the area even at a later date.

Nevertheless we have local examples of «high flame» kilns with this same kind of furnace chamber with upper feeding (like the Abbasid pottery kiln found also at Amman Citadel, and feeded from two hoopers or chutes) that, on the contrary, would support this operational hypothesis. Similar samples can be found also in other areas of the Mediterranean basin, like the Early Medieval example from Crypta Balbi at Rome (Fig. 5). The latter consists of two separate sections: On the one hand, there is the big cylindrical lower furnace chamber excavated in the ground, 3m deep, lined with reused bricks, and pointed with clay as bonding/insulating agent. Also in this case it is very clear the perimetral step to built the stone «dome» over it,[3] but the mouth of the kiln is at the top of the furnace chamber (as in our case). On the other hand there is the upper chamber, built over the ground level, and almost lost. Nevertheless a sort of *praefurnium*, defined by two walls built with bricks

Figure 4
Jerusalem. Crusader lime kiln from Saint Anne Church

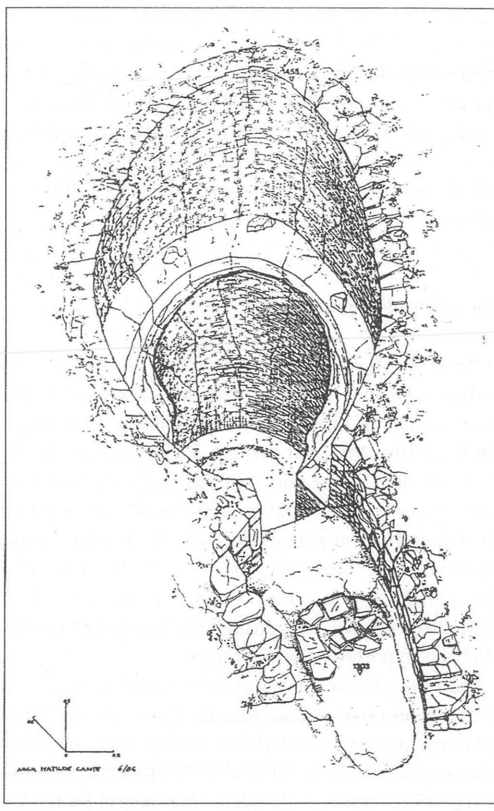


Figure 5
Rome. Early Medieval lime kiln from Crypta Balbi (Sagui 1986)

and stones is located at the mouth of the underground furnace chamber.

Also in favour of this first hypothesis there is the description of the way lime kilns were operated in Palestine up to a few decades ago. The description, according to a second hand account of how it used to work, corresponds to a kiln from a village near Ramallah, in use before 1967:

> A cylindrical ditch with a diameter of 3–4 meters and 3–5 meters deep is cut in the earth (see A in the sketch). A wall (E) about one meter thick and 1.25 m. high is built around the edge of the ditch. There must be two openings in this wall, a large one (C) 80 × 50 cms and a smaller one 40x30 cms. opposite to the first. The larger hole is used to supply wood to the fire while the smaller one serves as a smoke hole. The stone (F) to be burnt for lime is placed in several layers on a roof *[sic]* (G and H). The roof is formed of *[supported by]* long poles of wood (B) which support a layer of branches (G) over which is placed a layer of mud (H) at least 10 cm. thick [!]. The stones to be burnt are laid in a dome without mortar so that they will not fall into the pit after the wood is burnt. This same method of dome construction can still be seen in old villages houses . . . (Khadijah 1971: 107).

However, this description present some problems, specially regading the thick mud bedding on top of which the stones would be placed. Most probably it is a mistake, as this would prevent the stones placed over it from being burnt. Most probably this layer of mud was placed on top of the pile of stones in order to insulate the whole and prevent heat losses.

Still, this operation process does not explain the (undesirable) massive presence of ashes in the standard Umayyad lime. The kilns with an opening at the bottom of the furnace chamber, appart from a more convenient fuel feeding, permit also to remove easily the ashes once the firing is finished, (and before the «dome» is dismantled), in order to prevent the eventual mixing with the quicklime. This would led to the idea that in the kilns with upper feeding furnaces, it is impossible to remove the ashes and consequently to prevent the «dirting» of the lime, but this is not the case, because an adequate cleaning can actually take place, and the lime lumps can be sieved to eliminate the bulk of the ashes (even a layer of sand can be laid over the ashes remaining before the quicklime dome is dismantled). An interesting description of the Roman lime kilns and the way they were brought into operation was recorded by Caton in *De Agricultura*.[4] In this treatise, among other comments, it is said that . . . «if a sole opening is to be used, a big cavity must be left inside, enough to contain the ashes, so that it would not be necessary to remove them out [during the process] . . . If two mouths or openings are used, no space left inside is required, when needed, the ashes will be removed from one of the openings, meanwhile the feeding will continue through the other one» . . . Unfortunately it is not clear the exact position (at the bottom or at the top of the furnace) of these openings.

The second hypothesis implies a different concept of firing the limestone: All the above described kilns carry out to the so call «high flame» firing process. These kilns are the standard ones used in places where plenty of wood is available to be used as fuel. The combustion relies on the high flames produced by the fuel, but a high loss of heat also takes place, requiring a continuous feeding of the (lower) furnace. There is a second kind of combustion process, the so called «low flame» one, that up to now was thought to have been devised just in modern times. In this case there is not a proper furnace phisically separated from the stuff to be fired (below it), that would require recurrent fuelling. On the contrary, in this second case, the limestone and the fuel are placed piled in alternative layers, burning toghether, being required just a small amount of oxigen provided by small flues. This proccess is much more efficient in terms of fuel consumption and temperature reached. Furthermore, in this process are used nuts shells, pine cones and olive pits as fuel instead of timber, because of its higher heat-producing power (these nutshells, etc, are mixed with some kindling and oil to ease its ignition). This is the principle of the kilns developed in modern times: The so called *mixed feed vertical continuous draw kiln* (or *continuous vertical shaft kiln*). These kilns have a cylindric shape (Fig. 6), and have at their base a mouth used just to remove the ashes and retrieve the quicklime once it is ready. Feeding is done continuously through the top of the kiln with the aforementioned alternative layers of limestone and fuel that flow down slowly, once the lowest layers are already cooked and removed. In the upper zone the mix gets heated; The central one is the burning zone, where the actual combustion takes place thanks to the poking hole at the base of this zone (This small hole provides to the mix the required oxigen for the combustion. In order to help this air

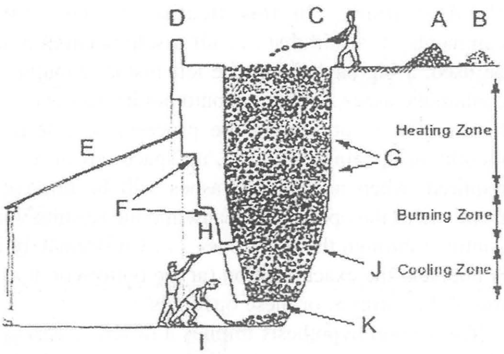


Figure 6
Modern mixed feed vertical continuous draw kiln (or continuous vertical shaft kiln). A: Lime for burning; B: Fuel (usually coal); C: Placing limestone and fuel in alternative layers; D: Parapet; E: Shelter; F: Arches; G: Alternative layers of lime and fuel; H: Poking hole; I: Burnt lime being removed; J: Brick or stone lining; K: Grate

entering, a long iron pole is introduced to poke the fire, opening cracks in the mix); The lowest section is the cooling zone, that ends at the bottom where the mentioned opening for retrieve the final product is placed (this allow to separate the layers of ashes and slag, corresponding to the fuel, from those of quicklime). This continuos process permit also to keep a more efficient production pace, as it is not necessary anymore the cyclic routine of mounting the piles or «domes» of limestone over the furnace, and dismantling them once the firing process is finished, nor to ignite and extinguish the furnace at each cycle, with an evident gain in time and fuel (preventing also damages to the kiln itself, avoiding the troubles from this cyclic heating and cooling process). Limestone is preheated above the burning zone and cooled below it, while also warming the incoming air. Utilising the waste heat to preheat the stone, providing warmed air to the burning zone, and ensuring the quicklime is cool when discharged, significantly increases energy efficiency.

In our case the continuous supply of fuel and stone, and the related benefits of continuos operation, do not exist (being an «intermittent fire kiln»), but the «low flame» combustion principle and its energy-saving conception would be the same: Alternative layers of fuel and stone or marble, with just enough air entrance from the small flue located at the base, would give as a result the same kind of «low flame» combustion. Thus this kind of kiln still would offer a better energy efficiency than the traditional ones, as no timber wood is required, just nuts shells, pine cones and olive pits. These are actually the sort of ashes found mixed with the lime in the mentioned «grey Umayyad mortar». This undesirable presence of vegetal hygroscopic ashes finds thus a *raison d'être*, as in this case it would not be possible to separate the ashes from the quicklime. Just some lumps of clean lime could be retrieved separately, meanwhile the bulk of it would be unfailingly mixed with ashes. This would also reaffirm our hypothesis that this mortar would not be the result of a deliberated addition of a component with specific properties, but the result of certain production process that could not avoid the presence of several undesirable but unavoidable by-products in the lime itself. Furthermore this would mean that this «low flame» combustion process and the related piling-kiln to cook the lime would have been devised much earlier that thought.

Other kilns from Amman Citadel

The Abbasid pottery kiln from the area behind the congregational Mosque

This kiln was placed in the entrance corridor of an Umayyad building (Fig. 7) located at the eastern end of the north *ziyada* (perimetral street arround a Mosque) of the Umayyad congregational Mosque (Arce 2000 —in press—). Only remains the furnace chamber: It has a cylindrical shape, and was excavated in the ground (between the lateral walls of the mentioned corridor), and lined with stones. The upper chamber was supported by means of three parallel diaphragm arches E-W oriented, and made out of reused bricks looted from the *hypocaustum* of the Umayyad bath (the brick dimensions varies from 29 × 29 × 6,5 to 28 × 28 × 6 cm, Arce 1999 —in press—). Only the springers of the southernmost arch still survives (Fig. 8). This lower furnace chamber was fed by means of two hoopers located one in front of the other, at the southern and northern sides. As in the case of the Umayyad lime kiln these hoopers tapper off slightly from the ground floor level. Any

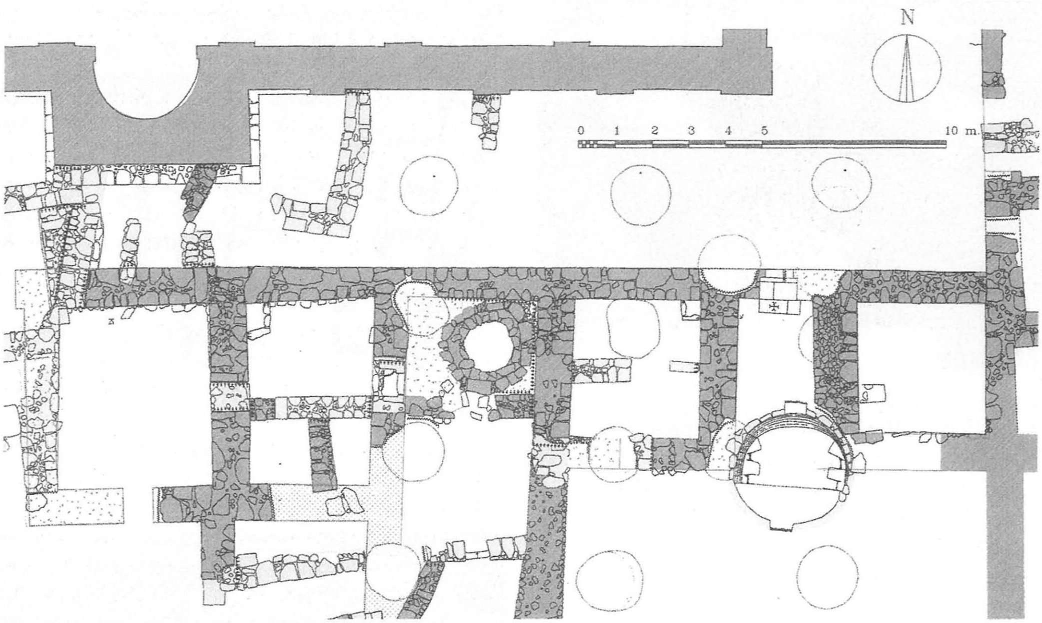
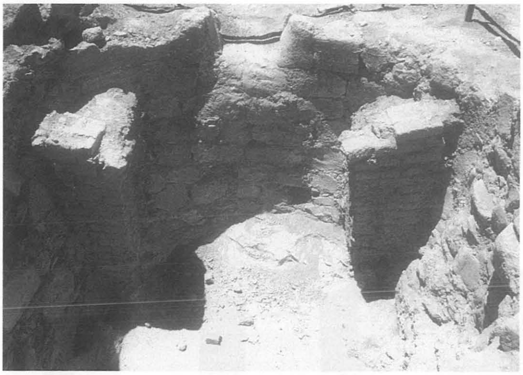


Figure 7
Amman Citadel. Location of Abbasid pottery kiln and Fatimid lime kiln

traces of the upper structure of the kiln have dissapeared. Nevertheless, it is quite clear that these arches supported the pierced floor of the upper chamber, where the clay objects (and eventually bricks) were placed for baking (Fig. 9a&b).

The Abbasid kiln from building «F» at the Umayyad Palace

This kiln was built in one of the buildings from the Umayyad palace itself, over the debris from the collapse of the structures that were destroyed by the 749 AD earthquake. It was built in the area were originally stood one of the *iwans* opened to the courtyard of the building. The original space was divided by two transversal walls, one at the edge of the court (blocking the *iwan*), meanwhile the second divides in two the original room (Fig. 10). Between both walls was placed the kiln and a small room, probably devoted to store the fuel. It was never finished nor used. Due to this fact and to its unusal shape, is difficult to say for what exact purpose it was built. It was designed as a small semispherical dome (2,24 m of diameter) on a flat surface, and built with small square bricks, the dimension of which varies from 20 × 20 × 6,5 to 22 × 22 × 6,5 cm (Fig. 11). It is noteworthy that these bricks do not seem to be reused (as it is the case of the above described pottery kiln), but built *ex profeso* for this structure. The mouth of the kiln is defined by big reused ashlars of limestone. The threshold of this opening was placed approximately one meter high over the Abbasid floor level (more than one meter higher, in its turn, than the original Umayyad floor, due to the deposit of debris). This threshold height is more or less the same of modern western bread kilns. This fact, besides its shape and the small dimension of the furnace chamber, could mislead to identify it as a bread oven, but this is, by no means, the case.[5] Most probably, it was an industrial furnace, for the production of metal or glass, although there are not enough evidences to assert undoubtedly which of the two choices is the correct one.



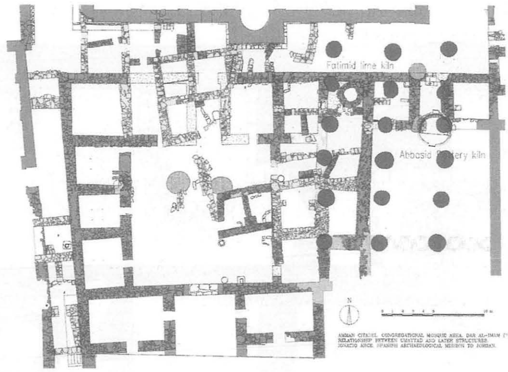


Figure 8
Amman Citadel. Abbasid pottery kiln. General view.

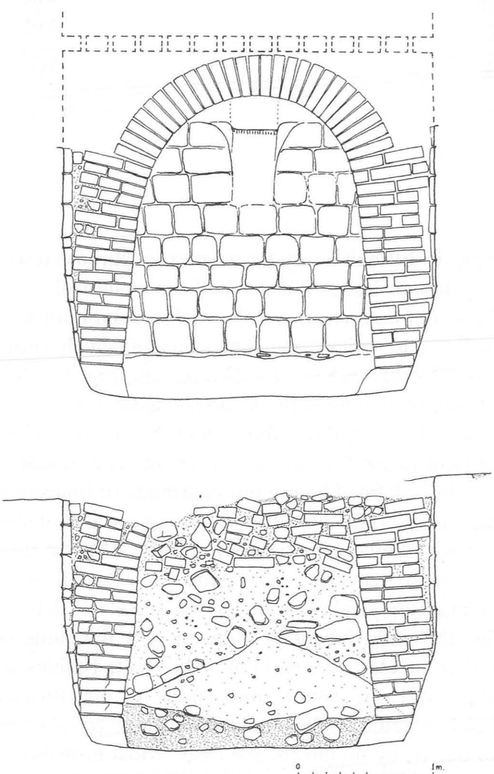


Figure 9a&b
Amman Citadel Abbasid pottery kiln. Sections. Present state and restoration

Figure 10
Umayyad Palace of Amman. Abbasid kiln from building «F» .General view

Figure 11
Umayyad Palace of Amman. Abbasid kiln from building «F» . Detail

The Fatimid lime kiln from the area behind the congregational Mosque

Only the lower furnace chamber of this small lime kiln survives (Fig. 12) It was placed in the corner of one of the rooms from the same building near the Umayyad congregational Mosque, where the Abbasid pottery kiln above described, had been found. When the former was built the latter was apparently out of use (the stratigraphy that would have related both structures was badly disrupted by the foundations of a modern concrete store built over them, see Fig. 8). It was built within the room defined by a late (Abbasid) wall that divides the original Umayyad room in two sections, so that it was contained by perimetral walls, being the space in between filled with earth. Actually, several lime kilns have been found not excavated in the ground, but built within previous existing built structures, like towers, or rooms, to offer a better insulation, and reinforce their stability, strenght and cohesion. The kiln furnace consists of two concentric rings of irregular stones masonry bound with earth: The internal one (1,3 m inner diameter; 0,9 m high) is lower than the external one (1,9 m inner diameter; 2,7 m external diameter; 1,4 m high), so that it defines the perimetral step required to built over it the «dome» of stones to be burnt (Fig. 13). At the bottom of the southern face of

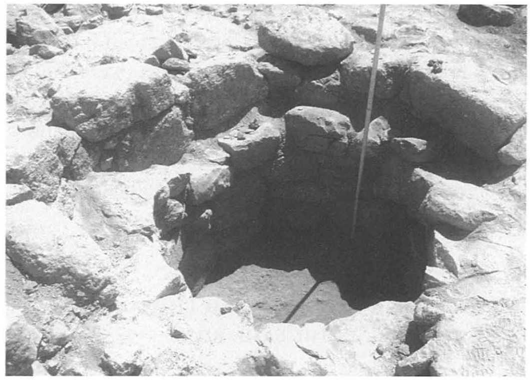


Figure 12
Amman Citadel. Fatimid lime kiln from the area behind the Mosque. General view

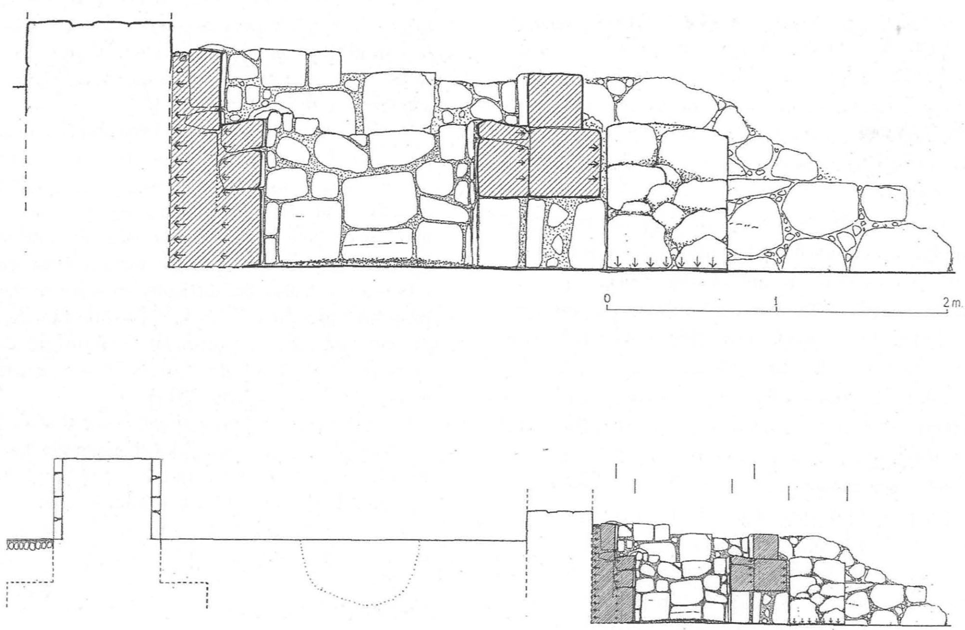


Figure 13
Amman Citadel. Fatimid lime kiln. Section

the furnace wall (in correspondence with the door of the Abbasid wall, there is the mouth to feed it (42 cm high; 30cm wide). Inside the chamber were found ashes and traces of lime, and partially burnt limestone. Among the pottery sherds found, a sample from Ayyubid-Mamluk period placed at the bottom of the kiln, on top of the ashes, would indicate the last period of use of the kiln.

Annex: Shared production of lime and bricks/pottery in the same kilns

In western medieval Europe, we know from written sources (legal norms and regulations) that the same kilns devoted to brick fabrication were producing lime as well. At Florence, during the medieval ages and the *Renaissance*, bricks and lime could be produced in the same furnace. The regulations of 1325 and 1415 from Florence make explicit the duty of brick kilns to produce lime as well. Furthermore the register documents of the related guild (*l'Arte dei calcinarii*) from the same city, do not make differences between the furnace workers of lime kilns from those from brick ones, as it used to happen in Bologna (Goldthwaite 1984, 267 cit. in Baragli 1998, 126). It make sence that due to economical reasons the same procedure would have been in use in the Levant. J. P. Adam noticed that in some areas of present day Tunis (Kairouan and Nabeul) the baking of bricks and lime is often made simultaneously, placing in the kiln («high flame» one) the bricks on top of the lime stones to be burnt. He mentions that no archaeological evidence, nor historical account of the use of this procedure in the Antiquity is recorded (Adam, 71; Fig. 154).

Yet, the «low flame» method, more energy-efficient, for lime production that apparently was devised in the Levant, would not allow a simultaneously baking of bricks. However, the same kiln could have been used for producing lime and baking bricks alternatively, just by changing the way the furnace was feeded and operated (in a similar way our Umayyad lime kiln might have been reused for melting metal in Fatimid period).

In our case, we have the evidence of two different kilns simultaneously in use, for lime and pottery production respectively (the first is Umayyad reused in Abbasid period, and the second one Abbasid). Apparently, it seems that each one had a very specific aim, and consequently a different design and operational system: The one for the lime would be a «low flame kiln», very adequate and efficient for lime production, meanwhile the pottery one would be a «high flame» kiln, required for pottery baking (the «low flame» ones cannot be used for this purpose). This would indicate that the bricks eventually baked at Amman Citadel during the Abbasid period should have been produced in the second kiln, or in the first one but using it as a «high flame» kiln. But they never could have been baked at the same time the lime was produced, if the mentioned «low flame» process was the one actually used.

Notes

1. The charcoal and ashes encased in this kind of mortar enables to date it with great accuracy by means of radiocarbon dating (Berger 1992; Gallo 1998). In collaboration with Dr. Ferran Alonso, and his team from the Institute of Physical Chemistry «Rocasolano» (CSIC), it has been foreseen the possibility of establish a research program to analyze and dating the samples collected by the Author. This would allow to test and calibrate the instrumental dating process, and at the same time to have a certain dating of these monuments. It must be taken into account that most of these Umayyad monuments were built in a very short and well defined period of time: the last years of the 7th C. and the first half of the 8th C. AD.
2. J. P. Adam distinguish three different kinds of operation process for burning the limestone: The kiln with furnace in the base («high flame» burning procedure), the «piling» kiln («low flame» burning procedure), and burning the lime in the open air. Regarding the second method («low flame»), Adam mentions the energy-efficiency but also the difficulty to separate the lime from the ashes, and the lack of known examples from the Antiquity. The last procedure, can just be used for gypsum production, due to the low temperatures reached. (Adam 1984, pp. 73–5).
3. «Il tratto superiore della muratura, che doveva essere costruito fuori terra, non è più conservato. Le pareti della zona inferiore della fornace sono costituite, per una altezza di cm. 170 circa dallo spiccato, da una muratura più spessa di cm 40 circa rispetto a quella della zona superiore. La differenza di spessore tra le due parti dà origine ad una risega che, avanzata verso l'interno della struttura, ne segue la intera circonferenza. La zona compressa tra la base della fornace e la risega, costituisce la camera di combustione della fornace L'alimentazione della fornace avveniva da

una zona posta inmediatamente all'esterno e comunicante con la fornace stessa: il prefurnio, costituito da una superficie ovale leggermente depressa e di modeste dimensioni (cm. 250x130 ca.), posta ad un'altezza quasi corrispondente alla risega della fornace (Sagui 1986, pp. 345–9).

4. The original text says: «Fornacem calcariam pedes latam X facito, altam pedes XX, usque ad pedes tres summam latam redigito. Si uno praefurnio coques, lacunam intus magnam facito uti satis siet ubi cinerem concipiat, ne foras sit educendus, fornacemque bene struito; facito fortax totam fornacem infimam complectatur. Si duobus praefurniis coques, lacuna nihil opus erit; cum cinere eruto opus erit, altero praefurnio eruito, in altero ignis erit. Ignem caveto ne intermittas quin semper siet, neve nectu neve ullo tempore intermittatur caveto. Lapidem bonumin fornacem quam candidissimum, quam minime varium indito. Cum fornacem facies, fauces praecipites deorsum facito; ubi satis foderis, tum fornacilocum facito, uti quam altissima et quam minime ventosa siet; si parum altam fornacem habebis ubi facias, latere summam statuito aut caementis cum luto, summam extrinsecus oblinito. Cum ignem subdideris, si qua flamma exibit nisi per orbem summum, luto oblinito. Ventus ad praefurnium caveto ne accedat; inibi austrum caveto maxime. Hoc signierit ubi calx cocta erit: summos lapides cocto esse oportebit; item infimi lapides cocti cadent et flamma minus fumosa exibit» (Caton, De Agricultura XLIV,1–4 *De fornace calcaria*).
5. The typical unleavened bread of the Middle East, is baked since the Antiquity in small clay made ovens: The *Taboun* and the *Tannour*. In the first case the oven (*Taboun*) has a rounded spherical shape that houses inside it the fire (feeded through an small opening at the base of its lateral wall). In this case the thin forms of unleavened bread are baked by placing them on top of its external surface, being the most widespread and usual method. The second kind of oven *(Tannour)* is slightly bigger and has a cilindrical shape opened at its top, housing the fire inside it at its base. It is used for cooking with pots or pans that are placed on top of it. Eventually (although quite rarely) it can be also used for baking bread: In this case the forms of thin bread are baked being adhered against the inner face of these ovens. More conventional stone built ovens exist: The so called *furun*.

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From the diaphragm arches to the ribbed vaults. An hypothesis for the birth and development of a building technique

Ignacio Arce

The aim of this paper is to discuss, on the evidence of the vaults and roofing systems present in different structures built during the Umayyad period, specially those from Qasr Harane, the birth of the first ribbed vaults as a development of the diaphragm arch roofing system, that will be latter developed in Al Andalus (Bab al Mardun mosque, Cordoba mosque, Vera Cruz and Torres del Rio churches, etc), and in the Transoxus-Khorassan Region (Sultan Sanjar mausoleum at Merv, etc). Besides, important remarks about the building techniques used in the Middle East for the construction of these arches, and relevant for understanding their structural and design concepts are also presented.

Introduction. The diaphragm arch. Origins and development

The so called «diaphragm arches» are first found in Parthian architecture, as well as in the *Hawran* (the region between present day Jordan and Syria), although in an apparently later date. This system consists of a structural self-standing arch placed transversally in a room (thrown from wall to wall), intended to support a lintelled or vaulted roof, reducing the span to be covered by that roof in the longitudinal direction of the room. The arches can be placed in series of parallel rows, defining a sequence of regular subspaces or bays, covered independently (like the ones of the Umayyad mosques of Damascus and Cordoba). Eventually they will be also arranged in the two directions of space, giving birth to composite structures: The first cross-ribbed ceilings and vaults, the birth of which will be analyzed in detail in the following.

In the first case (arches supporting a lintelled roof), the bays so defined are short enough to be spanned by stone beams set close together which carry the flat

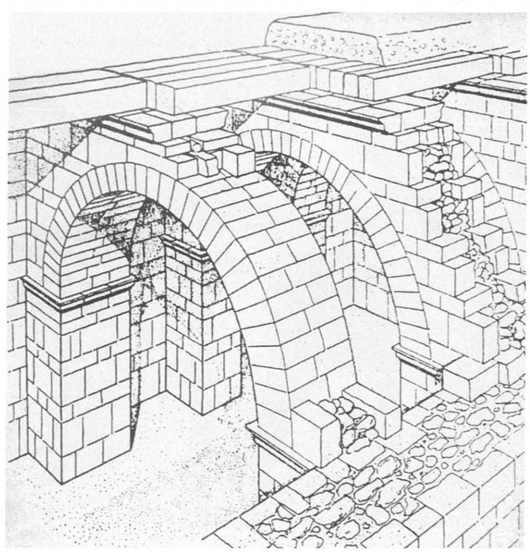

Figure 1
Hatra. Parthian Palace (Reuther 1938a: fig. 102)

floor (as can be seen at the Parthian Palace of Hatra for the first time. Reuther 1938a: fig.102 —Fig. 1—). This will give origin later to the medieval system of wooden pitched roofs and floors resting on these diaphragm arches (outstanding examples can be found in the *atarazanas* at Valencia, Poblet refectory, etc). The Roman architecture in the *Hawran* region (ancient *Auranitis*) presents numerous and conspicuos examples, as those found in the basilica at Shaqqa (Robertson 1985: p. 226 & fig. 99 —drawn by de Vogüe in 1875— Fig. 2), the temples at Atil, and in almost all the forts and castles from the *Limes Arabicus*. Other significative samples can be traced in singular and monumental buildings from the Byzantine (Fig. 3) and Umayyad periods confirming a continuty of use in the region throughout the centuries. It can be found also in hundreds of domestic houses from the Nabatean and Roman period onwards (for instance, at Umm al-Jimal or at Duma-Robertson 1985: p. 187–8 & fig.133, etc), becoming a traditional method, that will survive till nowadays (Marino and Lodino 1999) becoming the most characteristic building technique in Jordan and southern Syria.

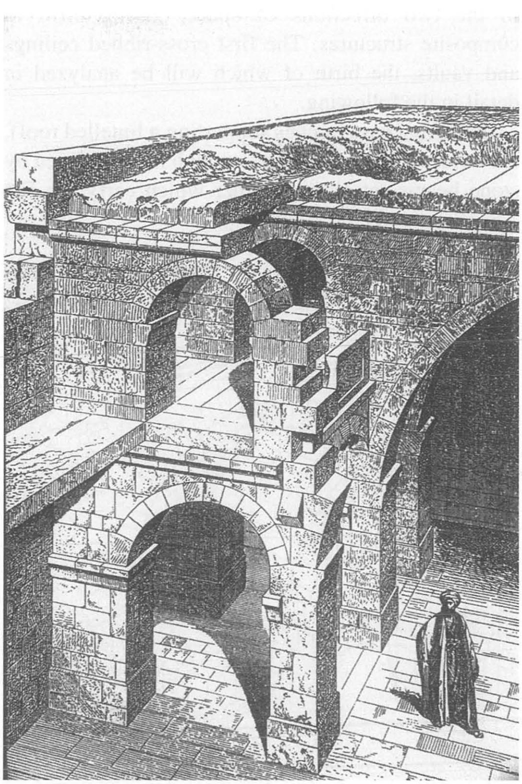

Figure 2
Shaqqa. Roman basilica (Robertson 1985: fig. 99 —drawn by de Vogüe in 1875—)

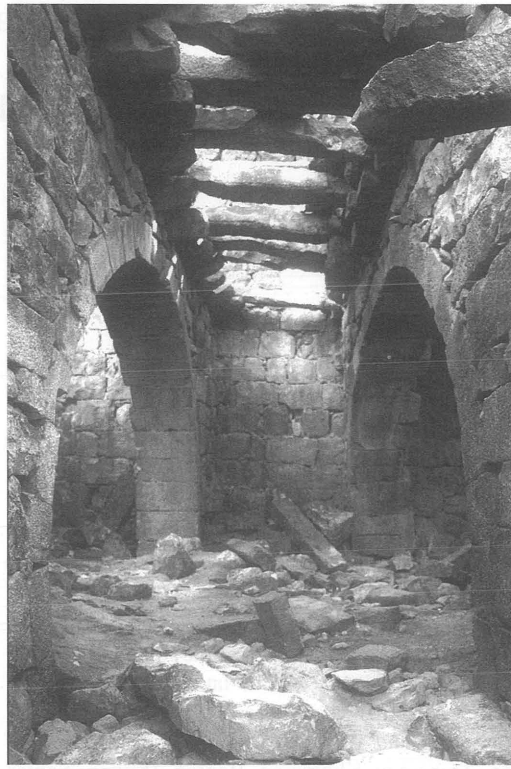

Figure 3
Umm Al-Jimal. Byzantine barracks

In the second case (arches supporting a vaulted roof), transversal barrel vaults are placed resting on series of parallel arches. The first samples recorded (unfortunately not surviving) would be the ones at the Parthian Palace of Ashur (Reuther reconstruction shows clearly its disposition (Reuther 1938a: fig.100 (Fig. 4), & Andrae and Lenzen 1933), as well as those from the Sassanian palace of Taq-i-Iwan at Kkark (also known as Iwan-i-Khark, dated by Herzfeld in the late 5th C.AD). Both, Dieulafoy and Reuther, reconstructed the latter with vaults spanning between

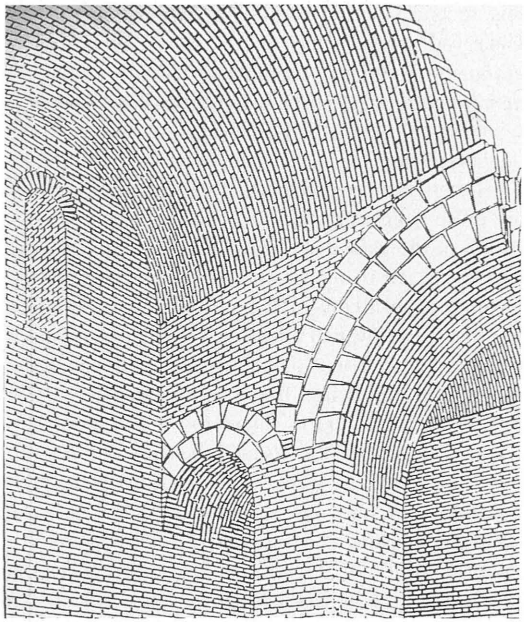

Figure 4
Ashur. Parthian Palace (Reuther 1938a: fig. 100)

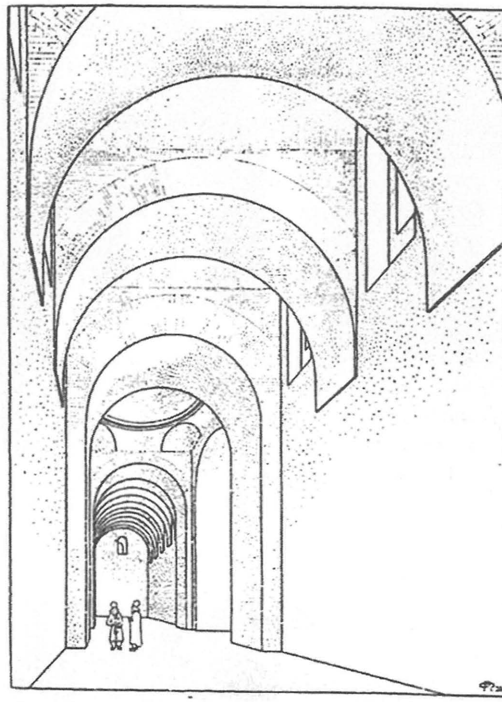

Figure 5
Khark. Sassanian palace of Taq-i-Iwan. (Reuther1938b: fig. 135)

the arches (Dieulafoy 1884: pp. 79–88 & Figs. 55–62; Reuther 1938b: fig. 135 Fig. 5). It would be found also at Sarvistan Palace (Reuther 1938: pp. & figs. 151–2), where Lionel Bier suggests also this solution in his restoration of room 12 (Bier 1979: pp. 39–40). It is noteworthy that no example nor traces of this combination of barrel vaults resting on diaphragm arches, exist from Roman and Byzantine periods in the Hawran. Suddenly during the Umayyad period the system blossoms in Great Syria and several structures are built with it: Qasr Harane, the baths of Qusayr ‘Amra (Fig. 6) and Hammam As-Sarraj, the Halabat mosque and probably the audience hall at Mshatta. After the Umayyad period this solution dissapears in the region[1], but it continues in use in Mesopotamia at Ukhaidir, (a palatial complex built in the first decades of the Abbasid rule —late 8th C.AD.—) and in Iran, at the Tariq Khana mosque in Damghan, NE Iran, 8–9th C. AD —Fig. 7—), among other examples. In western architecture the only example is found at the Church of Saint Philibert in Tournous (10–11th C.AD. Dieulafoy 1884:V, p. 163 and fig.117 —Fig. 8—). The concept is undoubtedly of great relevance for the development of ribbed vaults, as it is the first case of raised vaults supported by arches.

More recently, Urice in his study about Qasr Harane (Urice 1987: p. 53) has posed the theory that

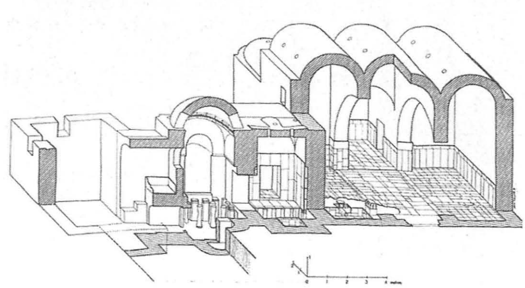

Figure 6
Qusayr ‘Amra. Umayyad baths

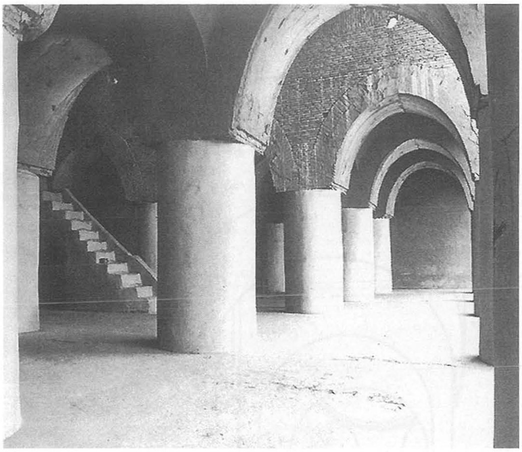

Figure 7
Damghan. Tariq Khana mosque

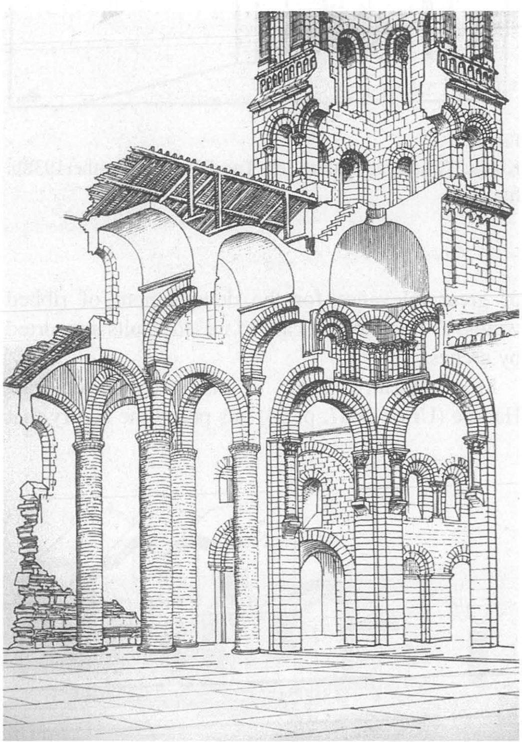

Figure 8
Tournous. Church of Saint Philibert. (Dieulafoy 1884:V, fig. 117)

this second solution would be an innovation from Umayyad period, due to the *«continuation and elaboration of an indigenous Syrian mean of monumental construction»* (Urice 1987: p. 54), rejecting its Parto-Sassanian origin because the doubts related to these first examples mentioned. Thus, he takes for granted that the earliest examples are those from Umayyad times, as he accepts Bier's dating of the palace at Sarvistan to the early Islamic period instead of the Sassanian origin suggested by Dieulafoy and Reuther. Regarding the two examples left, the Taq-i-Iwan at Khark and the Parthian Palace of Ashur, Urice points out that nothing is extant of the latter, and following Bier's opinion (Bier 1979: pp. 79–81), he rejects Dieulafoy's and Reuther's reconstructions of the former, because *«it has no basis on archaeological facts»* (Furthermore, Bier questions not only the reconstruction but also the dating, suggesting it belongs to the Seljuk period or later (!) —Bier 1979: p. 83—). Urice also points out the doubt posed by Godard: «Je ne suis d'ailleurs pas du tout certain que le coupe longitudinale d'Iwan-e Karkha ait été telle que Dieulafoy l'a dessineé. Rien n'indique en effet que ce bâtiment ait été voûte plutôt que couvert en terrasse, c'est dire qu'entre les arcs transversaux il y ait eu autrefois des voûtes plutôt qu'un plancher sur solives de bois» (Godard 1949: pp. 249–50). It must be pointed out that being Iwan-i-Khark a brick made building, it does not make sense to have been covered by stone slabs (traces of them should have been found among the debris), and in the case of a wooden flat roof, the span between two parallel arches should have been much wider than the actual one.

In my opinion, the barrel vaults on diaphragm arches scheme would have been introduced from Persia or Mesopotamia[2] (where it would have been developed and used for a long period) into the Hawran during the Umayyad period toghether with many other building techniques and materials (evidences of an intense interchange of building techniques exist during this period —see Arce 2000; Arce 2001; Almagro and Arce 1996).

On the one hand, it must be taken into account the continuous, although intermitent, cultural interchange that has existed (specially from Alexander the Great times onwards), between Mesopotamia and Syria across this border region, being the early Islamic period one of the most significative and intense of

them[3], as during that period the actual political frontier ceased to exist. Certainly Umayyad architecture and building knowledge took benefit from the conscript workforce brought together by the new rulers from the newly conquered territories of Persia, Mesopotamia, Syria and Egipt. This «melting pot» of technicians, architects and artisans would give birth, by means of mixing different architectural typologies, building techniques and decorative patterns and concepts, a brand new art, specially in architectural grounds (Arce 2000 & 2001). But in this case the building system seems just to have been introduced, not devised, in Syria during that period: It does not make sense that a brand new technical innovation, would be found in ALL the rooms of the very first building that makes use of it (as it occurs at Qasr Harane, the earliest Umayyad building using this technique), without any hesitation in its execution, that should have been the logical result of such an experimental process. Unexpectedly, they are built with a very high level of perfection, just achievable as the result of a well established and standarized procedure (compare with the «experiment» of crossing two diaphragm arches: it is carried out in a sole room, and in a quite awkward way —see below—).

On the other hand, all the existing evidences suggest a Mesopotamian/Iranian origin for both techniques (lintels and vaults on arches), as well as for the diaphragm arches and the barrel vaults themselves (the origins of which can be traced back in Mesopotamia during the Babylonian period[4]). The Parto-Sassanian origin of the diaphragm arch is also clear, as the oldest samples known are from that period & region (Ashur, Hatra, Kharkh), being probably introduced into the Hawran region during early Roman times (the examples from Assur and Hatra have been dated without discussion back to the Parthian period), meanwhile no example earlier than Nabatean-Roman Period has been found in the Hawran or in other places in Syria.[5] If both technical improvements are from Masopotamian/Persian origin, it make sense that the combination of both, would have been also devised in that region.[6] Besides, the system continues to be used in Iran and Mesopotamia after the Umayyad period, meanwhile it ceased to exist in Syria after the fall of the Umayyad rule.

The Umayyad examples. The case of Qasr Harane

This isolated Umayyad «desert castle», placed 80Km. to the East of Amman in the Syrian desert is, for several reasons, a building of extraordinary architectural innovation. Apart from other significative constructional features (semi domes on squinches, prefabricated elements in gypsum, etc. —see Urice 1987 & Arce 2000—), it is the most outstanding building regarding the use of barrel vaults on diaphragm arches, as almost all the rooms were covered with this system, and because it is certainly the earliest among all the Umayyad examples (consequently the oldest sample still surviving in Syria) to use this method.

Regarding the construction of the diaphragm arches themselves at Qasr Harane, two different building

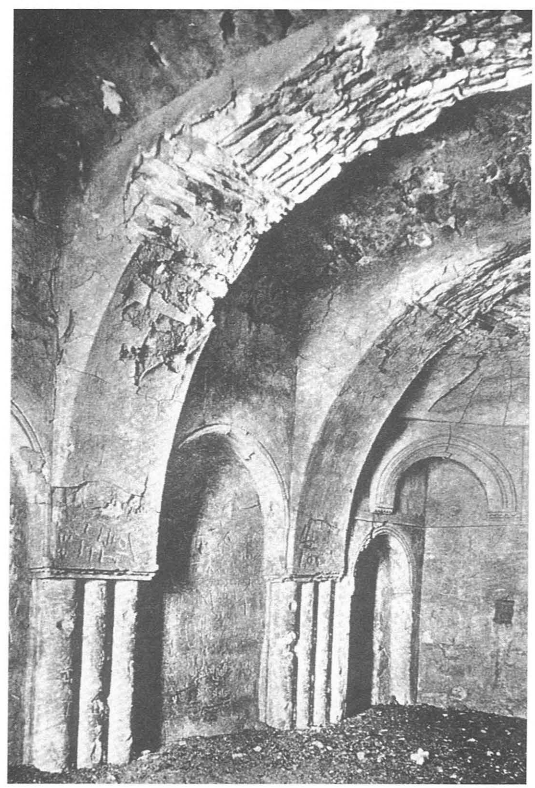

Figure 9
Qasr Harane. Arch building technique I

techniques can be noticed. In both cases roughly cut flat limestone voussoirs and gypsum based mortar are used: In the first case, the springers of the arch are built without any centering, setting the stones flat in projecting radiated courses (*«por lechos»*, i.e. «horizontally», or more precisely, parallel to the axe of the room) up to cover the correspondent extent of 1/4 or 1/3 of the span), meanwhile the central stretch is built placing the stones vertically perpendicularly to the axe of the room (*«por hojas»*) (Fig. 9). In the second case, meanwhile the springers are like in the previous one, the central stretch is built with the help of two lateral permanent ribs, «forms» or «centerings» of precast gypsum, that help to continue raising the arch without a traditional centering (just a light support to keep these pieces in place) is needed. These elements offer the required temporary support to the new courses (that are built leaning against them) and help to define the desired profile of the arch, working thus also as a form. Once the arch is finished they remain embedded in the structure (Fig. 10).

Both systems are from Mesopotamian/Persian origin: The first one is already found in the Parthian palace of Ashur (although there are used bricks, instead of flat stones —see fig. 4—). The second one is found throughout Persia, in the Umayyad Palace at Amman Citadel (Fig. 11 Arce 2000: p. 118–20 & figs. 14a & 15 and Almagro and Arce: pp. 28–9 & Fig 6), and also at Ukhaydir (Fig. 12 —from Reuther 1912—). In Ukhaydir the two permanent pre-cast ribs span all the width of the opening (similar, but smaller samples of full-span precast ribs can be seen at Harane itself, in

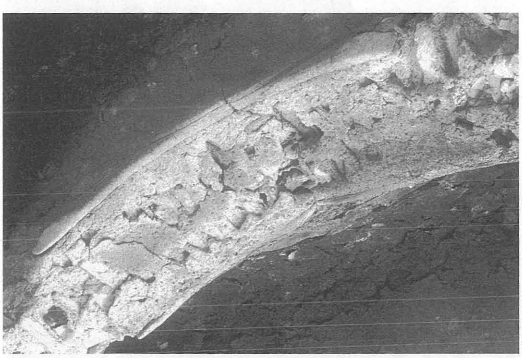

Figure 10
Qasr Harane. Arch building technique II (Gypsum precast embedded ribs)

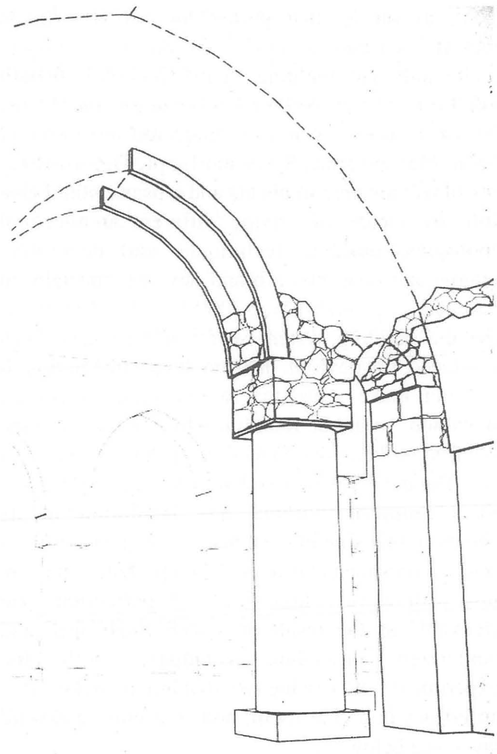

Figure 11
Umayyad Palace at Amman Citadel (Gypsum precast ribs)

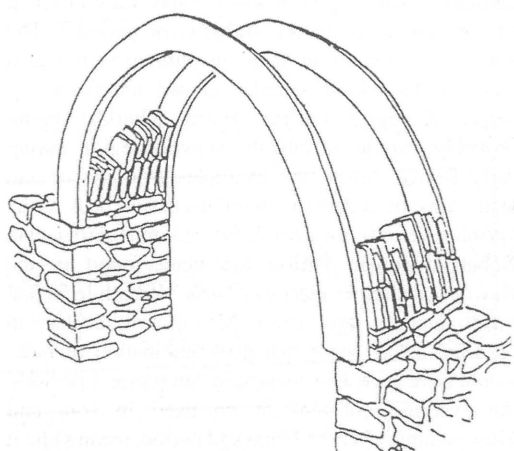

Figure 12
Ukhaydir. Gypsum precast ribs (Reuther 1912)

the row of small windows at rooms 29 and 59, built with these elements —Arce 2000: fig 14a—).

The Vaults are built following the Mesopotamian origin method of rings of vertically laid courses leaning against the end walls of the room to be roofed («por hojas»). The space left in the central area is covered following the same principle, just turning 90º the way the stones/bricks are laid (see Fig. 9). In our case instead of bricks, flat limestones are used.

The cross ribbed ceiling of room 61 at Harane. The genesis of the ribbed vaults

Room 61 is the most unusual chamber of Qasr Harane, it just measures 3.50 by 3.90 and is covered by an extraordinary combination of two diaphragm arches displayed perpendicularly to each other. They spring from the midpoint of the walls of the room from a triple recessed corbel, giving as a result a crossed structure that divides the ceiling into four square areas covered by that support four sets of coffers (Fig. 13). These coffers, also with a clear Parto-Sassanian origin, consist of recessed squares rotated at 90º, and are similar to the ones supported by the squinches in room 51 (Urice 1987: fig. 25) and to those from Amman Citadel Throne Hall (Arce 2000). Jaussen & Savignac suggest that the innermost part of the coffers could have housed an small dome, but taking into the account the antecedents and parallels of those coffers, this hypotesis does not seem to have a sound basis.

The importance of this ceiling is capital for the study of the birth of the ribbed vaults as it would represent the earliest antecedent of the ribbed vaults that later are to be found in Spain, Armenia and in the Transoxus region. It is actually the «missing link» that relates undoubtedly, the cross ribbed ceilings and vaults to the diaphragm arch technique, clarifying and demonstrating their origin.

Before the restoration carried out in the 80's by the Department of Antiquities, it was clear (Fig. 14a&b; Urice 1987: figs. 37–8) that for the construction of

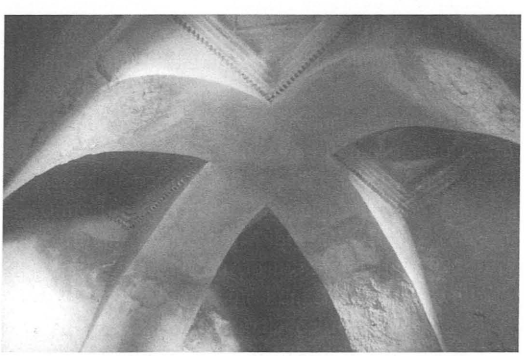

Figure 13
Qasr Harane. Room 61. Cross ribbed ceiling. Present state

Figure 14a&b
Qasr Harane. Room 61. Cross ribbed ceiling. Before restoration (Urice 1987:figs. 37–8)

these arches it had been used permanent precast gypsum ribs («embedded centerings or forms»), being set first one of the arches, and immediately after the second one, that consists of two sections leaning against the crown of the first one.[7]

Development of the system

The next step in the development of this design concept led to cover a square room with two pairs of parallel arches crossing each other at 90º, instead of single ones. They can be placed parallel to the walls of the room or diagonally, springing from adjacent walls. At the *Tornerias* mosque in Toledo (2nd half of the 11th C.), it can be found an outstanding sample of the first solution, that divides the ceiling into nine square sections or bays, that on their turn, are covered by pairs of single arches crossing at 90º springing from the mid points of the square bays (as in Harane), or from the corners.

Increasing degrees of sophistication are achieved when both possibilities are combined and eight arches arranged in four pairs, define an eight point star (two rotated squares). The arches can also span from the corners of the room to the midpoint of the opposite walls, giving as a result pairs of arches parallel but not in axe, that are interlaced creating an interesting tridimensional braid effect (see the SE bay vault of Bab al Mardun mosque in Toledo. Almost all the possible combinations (Fig. 15) can be found at this well known mosque from the 10th.C. AD (up to now it was the very starting point of this «chain» of examples): It offers an incredible catalog of solutions that are even combined one atop another (S, SW, NW & NE bays). More refined in their execution are the examples from the *maqsura* at the Corboba mosque (belonging to the enlargement commissioned by Al Hakam II —also 10th C. AD— Fig. 6).

The first Christian building in the Iberian peninsula that presents this solution is the Vera Cruz church at Segovia (12th.C.AD): The stone vault of the upper central chamber of this outstanding central-plan church is covered by a rather ackward and crude solution consisting of two pairs of ribs crossing at 90º quite close to each other (Fig. 17). It can be seen that the second couple of arches abut on the first one (as in Harane). The next step, that foreshadows gothic solutions, is found at the Holy Sepulcre church in Torres del Rio

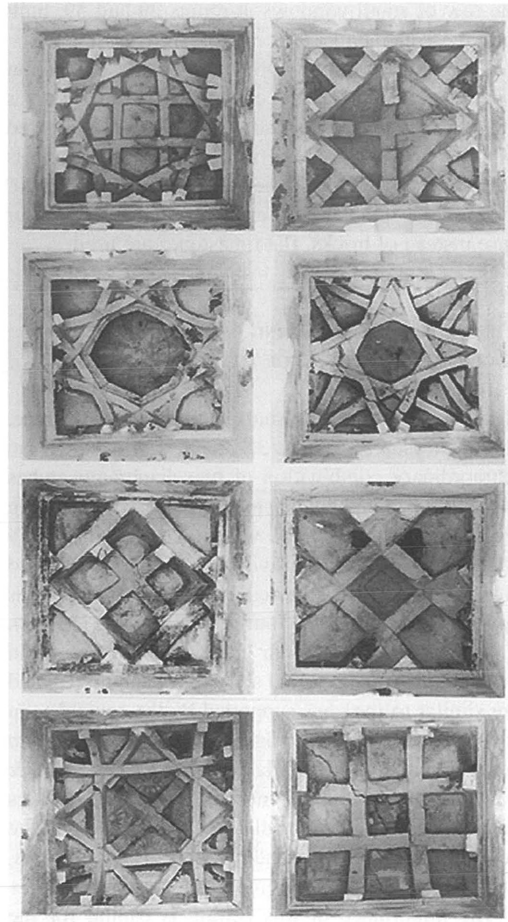

Figure 15
Toledo. Bab al Mardun Mosque. Ribbed vaults

(late S.XIII): It was built with pointed arches (Fig. 18) and using a carefully cut and dressed stone elements[8], Several religious and civil *mudejar* buildings will make use of this system during the 12th, 13th.and 14th.Centuries (like the castles at Villena and at Biar —Ferre de Merlo 2000—). Later, during the Renaissance, Andrés de Vandelvira will recover and apply the concept of the two pairs of arches crossed at 90º at the Renaissance church of S. Ardrés in Ubeda. His son, Alonso, records in his treatise, the way of building it, existing some examples of this late Renaisance adaptation of the solution by other architects.

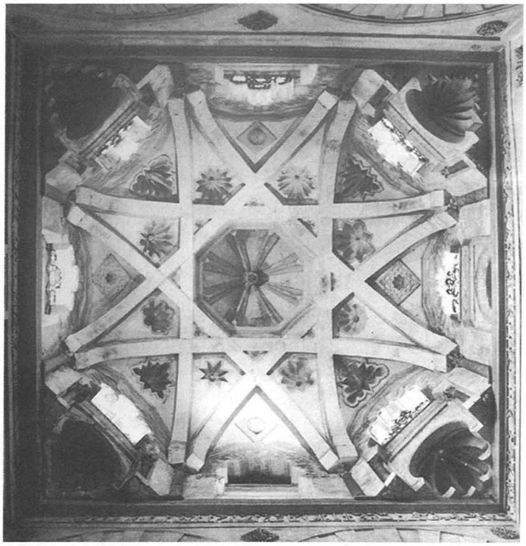

Figure 16
Corboba mosque. Ribbed vault from the *maqsura*

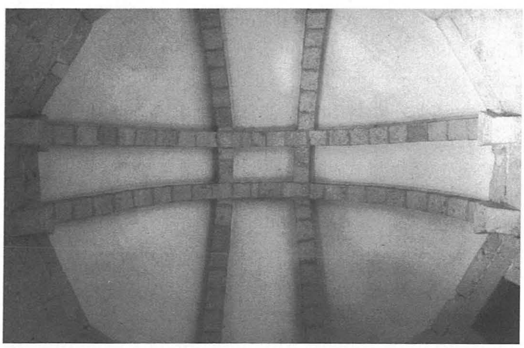

Figure 17
Segovia. Vera Cruz church. Upper central chamber. Ribbed vault

Several other vaulting traditions that stem out from this same concept deserve to be reviewed.

Timurid stellate vaults

It is likely that experiments with the simplest form of stellate vault gave rise to those of greater complexity. This is the cases of those found in the Seljuk and later

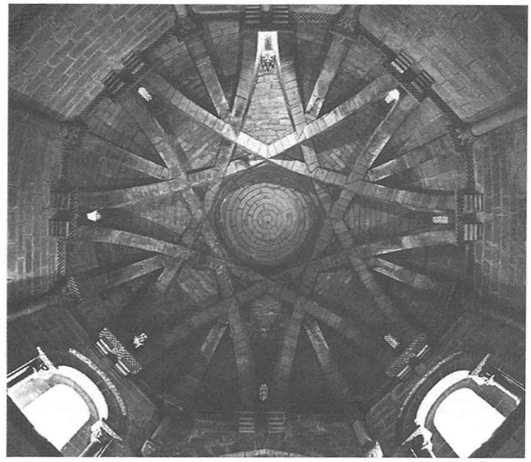

Figure 18
Torres del Rio. Holy Sepulcre church. Ribbed vault

in the Timurid architecture, where the number of these pairs of parallel brick arches rotating arround the centre of the room will multiply, giving birth to the so called «Stellate vaults» (Golombek & Wilber 1988: pp. 169–173 & figs. 42–45): *«The surface of the dome is broken up into multiple planes or facets, but the geometric scheme is preserved as a «skeleton» of arched ribs. These arched ribs interact within the pattern and become the arch net, filling pendentival areas, while delineating with their crowns a star poligon in the center of the dome»* (Golombek & Wilber 1988: pp. 169). The ribbed dome from Sultan Sanjar at Merv (Fig. 19) offers an outstanding sample of this development.

Gavits' vaulting

A singular case of ribbed vaults are those that cover the Armenian *«gavits»* (a kind of square narthex, placed at the western entrance of the Armenian churches, and used as an asambleary room —or *jamatun*—). In this case, four diaphragm arches displayed in pairs perpendicularly each other (and parallel to the walls of the square plan room), on which rest quarter sections of barrel vaults, defining a sort of ribbed cloister vault, with a central square uncovered bay, that gives light to the room (Fig. 20).

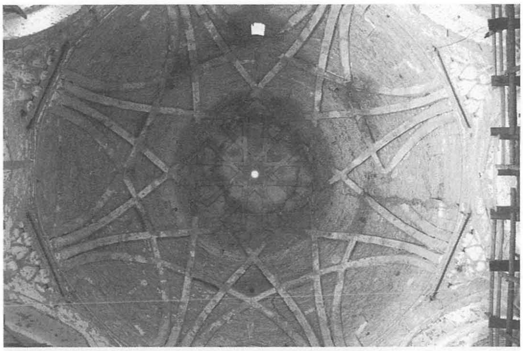

Figure 19
Merv. The ribbed dome from Sultan Sanjar mausoleum

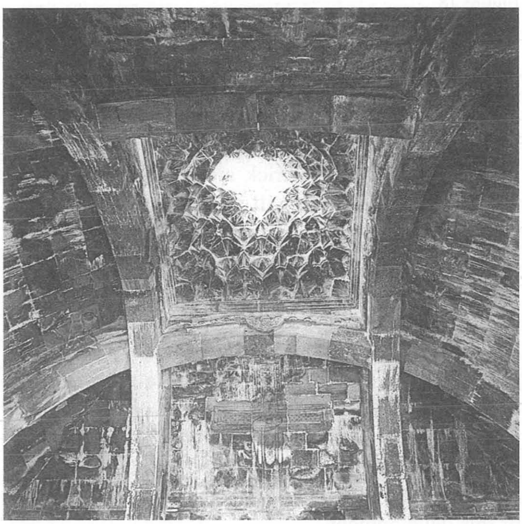

Figure 20
Gandzasar monastry (Azerbaidjan). Gavit's vault

As can be seen, it is a particular case of the simplest solution, but despite its simplicity (and because its clear structural and spatial concept), it will become a typical and distinctive structure of Armenian monastic architecture. Its design concept, and the date of the first samples dated (IX–X C. AD) suggest at least a parallel development, from a common root (Armenia before being incorporated to the Sassanian empire was a buffer state in between Bizantine and Sassanian empires).

Rib arches as temporary centerings. The case of the «*Quadripartite lanceolate vaults*» (pointed and ribbed cloister vaults) and the Roman cross groined «ribbed» vaults

Special attention deserve the analysis of a different kind of oriental «ribbed» vaults, as they provide significative information about a different concept in the use of arches for vaulting. This may offer an explanation for some details in the construction of actual ribbed vaults in Khorassan and Turan.

More loosely related to the vaulting concepts exposed up to now (and also different in building procedure) are the so called *«quadripartite lanceolate vaults»* (so called by Herrmann 1999: pp. 57–9 & 135) widely found in Central Asia and in the Khorassan, mainly during the Seljuk period (11^{th} and 12^{th} C. A.D.). Examples from earlier period may exist, like those from the Greater Kyz Qala at Merv (some authors date it back to the first Abbasid period —8^{th} to 9^{th}C.A.D.—) or at the Yakkiper Köshk at Merv, as well as in other places in the Khorasan region (the area nowadays between NE Iran and Turkmenistan) and in Turan (ancient name of the Centroasian region). The ribs that we found in these cases belong, as we will see, to the group of arches or ribs intended as temporary centerings of forms.

The usual building procedure of these brick (or adobe) vaults resemble (and combine) the one of the Sassanian *squinch vault,*[9] and that of the (also traditional Sassanian) parabolic-in-section barrel vault built without centering (Reuther 1938b: p. 499–500 & fig.129). Both, the *squinch vault* and the *«quadripartite lanceolate»* one, are built starting from the corners of the room to be roofed, without using any centering: In the case of the squinch vault, a series of small arches are placed diagonally across the corner of the room forming half-cone-shaped squinches that continue rising in a series of concentric brick courses, until the four half cones so formed meet up to form the vault itself (Fig. 21 —Reuther 1938b: p. 501 & fig.130—).

In the case of the «quadripartite lanceolate» vault, pairs of bricks standing almost vertically and leaning against each other are placed in the corners of the room. New and consecutive «rings» of vertical-placed bricks abut on the first ones, gaining each ring an increasing curved setting projecting inwards (as in the mentioned Sassanian parabolic barrel vaults). The result is a sort of «pointed» cloister vault, the

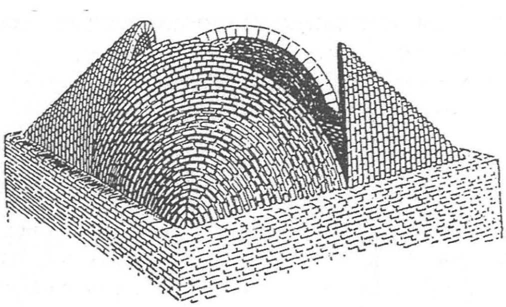

Figure 21
Squinch vault or *Balkhi vault* (Reuther 1938b: fig. 130)

geometric definition of which would be the intersection of two parabolic/pointed barrel vaults, instead of the usual intersection of semicircular barrel vaults (Herrmann 1999: fig.52).

When the span of the room is too big, pairs of reinforcing headers ribs (*a sort of header arches —«arcos de cabeza»—*) are placed near the corners in order to offer a better and more stable support to the next rings of bricks. So, after the first corner rings are put in place, a series of new ones are set projecting out a few centimeters from the former ones, defining the mentioned ribs.[10] In these vaults the pairs of ribs are placed close to the corners of the room, and they end where they meet each other, not being thus complete true arches[11] (Fig. 22). These parallel ribs are used as temporary support for the central section of the vault built in between them. This central stretch can be closed easily setting the remaining rings leaning against them: The result is a sort of ribbed cloister vault[12] of bricks set vertically that does not require any centering for its construction.

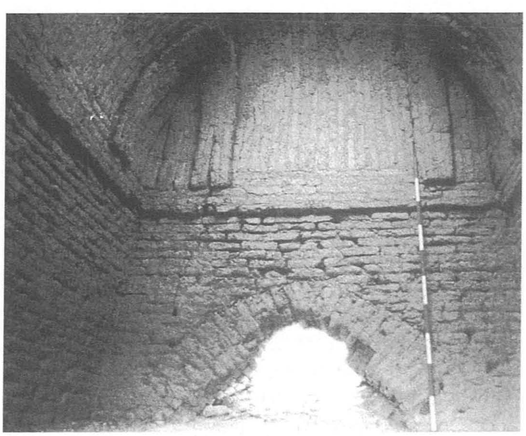

Figure 22
Merv. Yakkiper Köshk, room 6 . *Quadripartite lanceolate vault* (Herrmann 1999, fig. 51)

Discussion

This method proves that the conceptual improvement that can be seen in some Byzantine barrel vaults, had been already devised in the Persian region of Khorassan and applied for the construction of a more complicated structure. Thus, it would be the link between the Mesopotamian-Sassanian barrel vaults and those Byzantine ones that use embedded transversal arches in order to support the intermediate stretches of vault (that in their turn, are built with the —also Mesopotamian— method of rings of vertical bricks resting on these header arches —Choisy 1883— 1997: pp. 31–43 & figs. 30–41).

Conceptually this technique is much closer related to the above described Persian one of the precast gypsum centerings/ribs for the construction of arches (see above the «second solution» present at Harane, Amman & Ukhaidir). Consequently, it would be also conceptually related to the group of Roman «ribbed» cross groined vaults, as in all these cases the «rib arches» are not true ones but just a sort of embedded centering or form, intended as a temporary support during the construction, becoming afterwards part of the arch (or of the vault) itself, without an specific role, once the construction is finished.[13] Similarly, in the Roman concrete vaults the arches are embodied in the vault itself as intended merely as temporary supporting or coffering means during construction (Choisy 1873–1999 lams.VII–IX). This poses a series of interesting questions: Were the Romans aware of these oriental improvements? May these techniques have influenced somehow the development of the Roman technique of embedded rib cross groined vaults? The question remains open.

Appendix. Other Building techniques devised for arch construction in the Levant. The antecedents of the «Tas de Charge»

The aim of this section of the paper is to present the result of the research conducted on the building

techniques used for the construction of (diaphragm) arches in the Syrian region of *Hauran* from Roman to Islamic times. Earlier we have described in detail the methods for arch construction from Mesopotamian-Iranian origin, now special attention will be paid to the methods developed in this region during the Roman period An special stress is done on the peculiar shape and arrangement of the voussoirs of certain Roman masonry arches, and the way the centerings were designed and placed to built them. It will be shown how this method survived until the Umayyad period, and how it can be regarded as the technical antecedent of the «Tas de Charge», essential for the development of the Gothic ribbed vaults.

As in the previous cases, in order to reduce the extent that the centering must span when building an arch, the first courses or voussoirs are projected towards the centre of the arch, even in horizontal layers, without the help of any centering. Then, different solutions can devised to span the central gap: Previously we have described in detail the Mesopotamian-Iranian methods that can even avoid totally the use of a centering, by means of the use of precast gypsum ribs used as permanent centering/ coffer; now we will study the Roman ones devised in a region also lacking of timber to built wide-span centerings. In traditional Roman construction, the last voussoirs or courses of this lower section of the arch, often project towards the center of the arch further than the intrados profile of the arch, in order to support the centering required to complete the work. Relevant and well known examples of this are those from the *Pont d'Ambrussum*, or the *Pont du Garde* (Fig. 23) (Adam 1984: figs.420–1 & 662 and also Choisy 1999: P.112 & figs.77–78). This allow to reduce the dimensions of the centering as well as the timber sections required to built them. Besides, the direct thrust exerted on these special voussoirs, by the new upper ones placed with the help of the centering, counterbalance the one exerted by the centering itself, as both rest on that cantilevered projecting voussoirs. Once the work was finished these projecting elements were usually carved away, although in other cases (as the mentioned bridges) they were left in situ, due to the utilitarian nature of the construction.

In the Levant, a more sophisticated and efficient version of this method was developed in the Syrian region of *Hawran* during the Roman period (all the examples belong to the 2nd. C.AD). The method

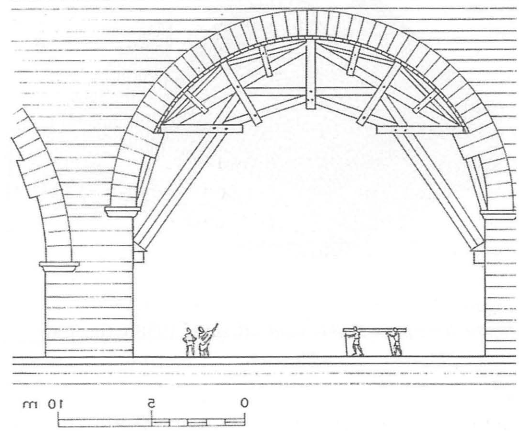

Figure 23
Pont du Garde (Adam 1984: figs. 420)

consists in embedding deep into the lateral walls (the spandrels of the arch) these special voussoirs, by projecting them also «inwards» (futher than the extrados profile of the arch). As in the previous case, it is not neccessary to use any centering in the construction up to the level of these special voussoirs, meanwhile for the central stretch remaining, just a small centering is required. This method will be kept in use during Umayyad period and extensively used in all kind of arch/vault construction.

A related antecedent of the design concept of this system could be traced in the Roman construction of lintelled arches without centering: Choisy points out the samples from the Verona amphitheatre (Choisy 1999 [1873]: p. 117), where the lintelled arch consist just of three pieces: the two corbelled «springers» embedded in the wall (the weight of which counterbalance the projecting section of the springer) and the keystone usually placed later. In our case the actual springer voussoirs that projects inwards and serve as support for the centering are also cantilevered pieces, being also embedded in the wall, and counterballanced by the weight of the courses from the spandrels of the arch (this system is also widespread in the *Hawran* region).

In these «oriental» cases that have been analyzed, although no traces of the «outer» projecting sections of the voussoirs survive (they were carved away

because belonging to civic and religious buildings and not to bridges as in the western samples), it is clear that these pieces were carved in this way, being afterwards cut away. These projections were just needed to support the centering during the construction of the central section of the arch, meanwhile the «inner» embedding offers a permanent and better bonding and consequently a better counterbalance to this cantilevered element, during and after the construction process.

Furthermore, this corbelled voussoirs (the uppermost of this lower section of the arch) is in some cases carved with a bent shape in order to improve ever more the bonding into the wall, offering at the same time a better counterbalance for the temporarily thrust of the centering. It also distributes better the arch thrust once the construction is completed and the centering dismantled, because the upper faces of this bent piece offer the ideal springer bed for both, the radial voussoirs of the upper part of the arch and the horizontal courses of the spandrels (see below). Another distinctive characteristic of these projecting voussoirs is that they are the only ones consisting of two adjoining pieces, placed one beside the other (due to its longitudinal shape and its corbelled function), instead of the single ones used in the rest of the arch voussoirs. It is interesting to note how these pieces work actually as a *«Tas de Charge»*, i. e. those lowest courses/voussoirs of a vault or arch laid horizontally and bonded into the wall offering an upper face with the required pitch to continue the arch. This arch springer, that will become essential during the Gothic period (as described by Viollet le Duc), has in these examples its more clear (and fully developed) antecedent .

The different building procedure used for the construction of the two sections of the arch can be also discerned from the clear difference between the way the voussoirs were cut in the first courses of the arch from those that conform the haunches and the

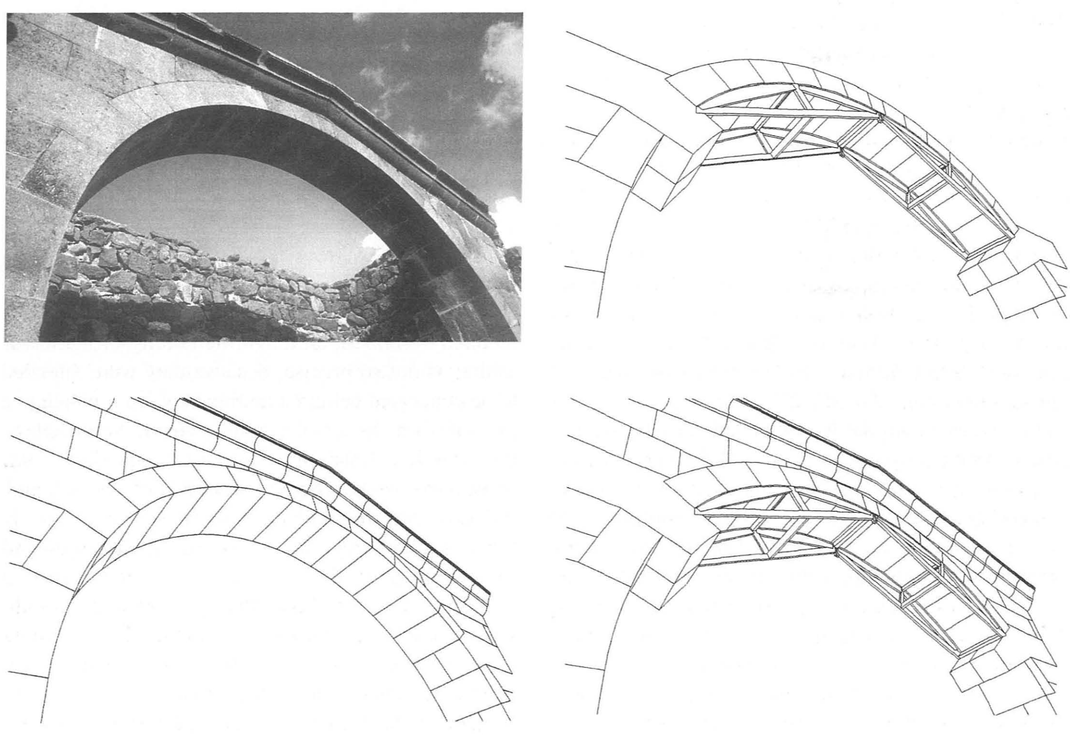

Figure 24a-d
Atil.North (Roman) Temple. Hypothesis of building process

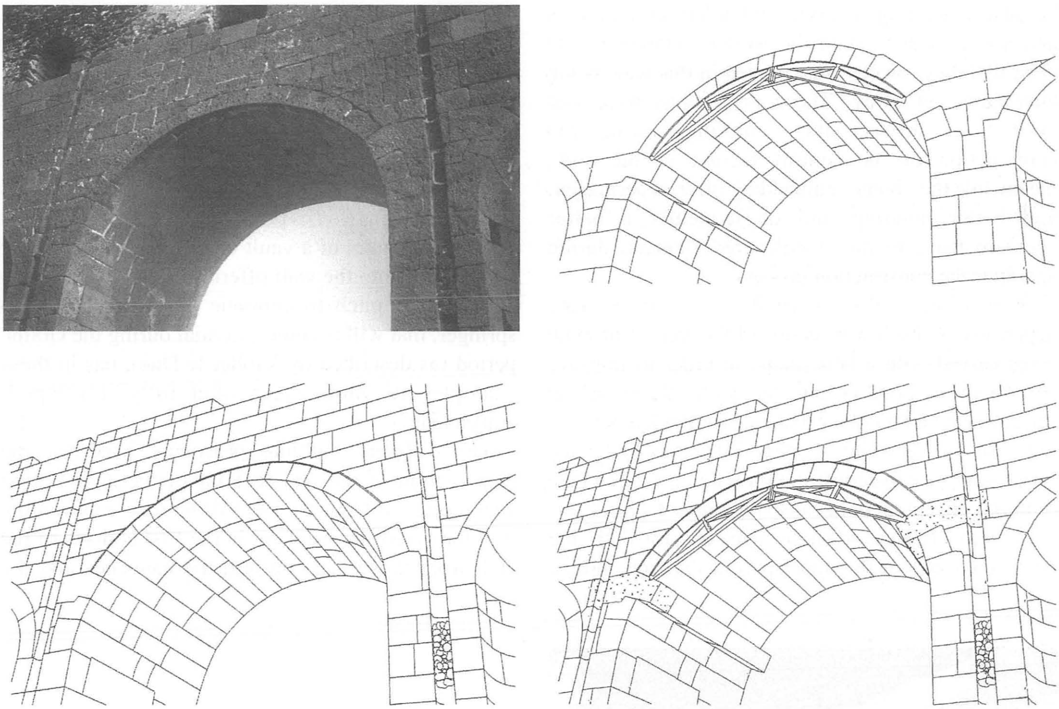

Figure 25
Shahba (former Philipopolis). Roman baths. Arch from the *frigidarium*

crown. The latter define a perfect extradosed arch profile, meanwhile the former usually have a stepped extrados, being interlinked with the masonry work of the wall fabric. Also a slight difference in the curvature of the intrados denote these two phases of construction (figs. 24a–d&25).

In the case from the North Temple at Atil, we can find a subtle improvement that toghether with the exquisite perfection of the dressing of its basalt ashlars, demonstrate the high level of sophistication and quality achieved, based on a precise construction efficacy in controlling the means and resources available. In addition to the mentioned features, the *«Tas de Charge»* voussoirs present in this case a shallow recess cut in the upper face of these special pieces, in order to prevent the sliding of the first voussoir of the upper-central section of the arch that rest on that piece and on the centering that springs also from it (fig. 24).

The examples from the Roman baths at Shahba (former Philipopolis) (fig. 25a-d) and Bosra (the capital of the *Auranitis* region), present the basic features described, although the cutting of the basalt ashlars is not so precise, because they were intended to be concealed behind a marble veneer, of which just the holes for the metal cramps remain. Nevertheless, this rougher finishing allow to notice better the mentioned slight change of curvature in the arch intrados, and the general concepts of the design. In these two buildings can be also traced the use of this same concept for the construction of semidomes.

In the Vestibule of the Umayyad palace at Amman Citadel can be found the adoption of this system in the construction of the four arches of the central space of the building: In this case the corbelled voussoirs project outwards further than the profile of the extrados and have a bent upper face, meanwhile the lower one follows the line of the radial joint (fig. 26).

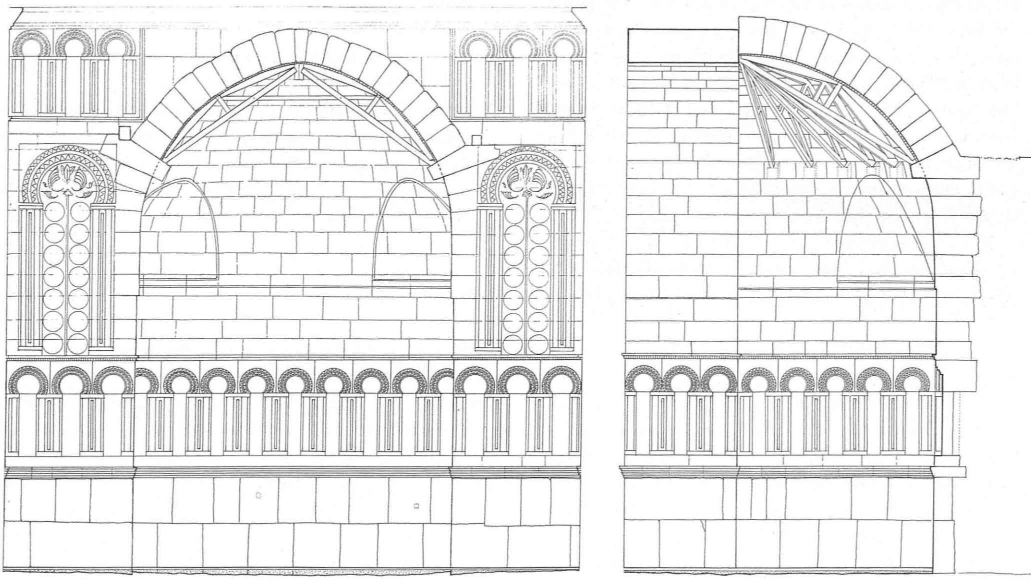

Figure 26
Umayyad Palace of Amman. Vestibule. Hypothesis of building process

There are evidences that prove that a similar system was also used for the construction of the lateral semidomes (as in the Termae from Bosra and Shahba): On top of the first horizontal projecting courses (where the «counterfait» squinches are carved —Arce 2000; Creswell 1969—), was built the upper section of the semidome (a «true» one, with proper spherical radial joints). This upper section was built springing from the uppermost course of the horizontal layers, that had been carved with the mentioned «Tas de Charge» profile, i.e.: offering an upper face with the required pitch to start the true dome (this was ascertained from the study of its extrados during the restoration of the monument carried out by the Author). The semidome abuts also on the above mentioned central arches (Fig. 27) and was built most probably on small centerings[14] that should have been supported by projecting sections of the mentioned transition course (that would have been cut away afterwards, as in the case of the arches).

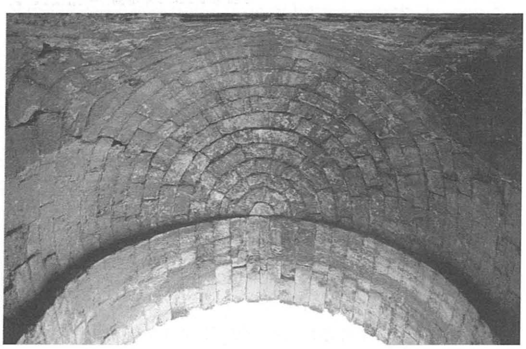

Figure 27
Umayyad Palace of Amman. Vestibule. Semidomes resting on horizontal corbelled courses (with counterfait or «false» squinches carved out on them), and leaning against central arches

Notes

1. For a detailed discussion (up to 1969) on the origin of this system, see Creswell 1969: pp. 444–49 & Godard 1949: pp. 244–50.
2. It is noteworthy that before an adequate assessment and dating were carried out, the first studies on Qasre Harane emphasized the «Mesopotamian» character of

the building in construction and decoration (Jaussen & Savignac 1922: pp. 114–21) or even postulated on the same basis a Sassanian origin (Dieulafoy 1913:15 and Warren 1977:49).

3. This can be traced very clearly for instance in the «classical» decorative patterns present in the Parto-Sassanian architecture since Alexander's period, that in some cases were «reintroduced» in Syria during Umayyad period (see Arce 2000).
4. Strabo in *Geographica*, Book XVI Chapter I,5 makes reference to the vaults that covered the houses in Babylonia and Susiana because of the lack of wood fit for roofing.
5. Nevertheless, Reuther follows the theory of Diez, suggesting that the diaphragm arch *«seems to have originated in southern Arabia and to have travelled from the Yemen north with the migrating groups of Azdites and Himayarites, who, when they settled in Syria and Mesopotamia, which was then under Arsacid control, introduced it to these regions, where, however, it came to be executed entirely in stone»* (Reuther:425 quoting from E. Diez, Die Kunst in der islamischen Völker, Berlin, 1915, pxii.). The fact is that no archaeological evidence exists to support this hypothesis nor traces of this technique survive in Yemen.
6. Besides, it must be pointed out that although Bier and Goddard reject skeptically Dieulafoy's and Reuther's reconstructions and dating of the earlier examples of vaults on diaphragm arches to the Sassanian period, they fail to provide sound evidences to support firmly their own hypothesis of dating. It is highly unlikely to have such an important development coming «out of the blue», meanwhile several evidences point out to the mentioned Mesopotamian-Iranian origin.
7. These photographs come from to the Rockefeller Museum photographic archive (at Jerusalem) and were taken in 1940 (Inventory #: RMP 23.149 & other two without inventory number).
8. It is noteworthy that both churches were built by the Temple order, which had gain a thorough knowledge of the oriental building techniques in the Near East.
9. The squinch vault is called *«balkhi* vault» in Central Asia, in reference to the city of Balkh (present day Mazar-i-Sharif) (see Herrmann 1999).
10. In other cases, the section of the vault varies in regular stretches defining series of raised ribs (and recessed areas), that in these cases run across the whole surface of the vault (see room 5 at Yakkiper Köshkh in Herrmann 1999: fig. 112).
11. These arches define its characteristic «lanceolate» shape, that led Herrmann to give this name to them (Herrmann 1999).
12. This led Pugachenkova, instead, to call them «monastery vaults» (for «cloister vault»). (Pugachenkova, 1958).
13. These ones require just a light support themselves during their construction.
14. In the construction of the traditional Jordanian houses, these small centerings required for the construction of the diaphragm arches, are still used, showing the pervivence of this technique in the vernacular architecture (Marino and Lodino 1999: fig. in p. 37).

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Dams from the Roman era in Spain. Analysis of design forms

Miguel Arenillas
Juan C. Castillo

A recent academic study undertaken by the authors[1] (Castillo 2002; Castillo y Arenillas 2000) has enabled the identification of remains of and references to seventy-two dams from the Roman era, constructed in Spain between the first and fourth centuries.[2] Fifty of them have been located and detailed. The twenty-two outstanding, although identified on the ground, have not been able to be acceptably characterized, due in some cases to their being ruins in a highly degraded state, others due to their being masked by repairs and reconstructions subsequent to the Roman era. A list of the properly characterized dams is enclosed in the Appendix at the end of this paper, where reservoir dams are illustrated on the one hand (21) and diversion weirs on the other (29). The main dams (nearly all of them are large dams by present standards) are included in Table I.

Location and emplacement of Roman dams in Spain

Most of the dams built in Spain by the Romans —and particularly the largest ones— can be gathered together in three main areas: the basin of the river Ebro, especially the right bank, whose focus may be located in Zaragoza (*Caesaraugusta*); the area of Mérida (*Augusta Emerita*) along the basin of the river Guadiana, and the left bank of the river Tajo in some points near Toledo (*Toletum*). The natural regulation of the rivers flowing in these sections of the Spanish mainland is low or very low, basically as a consequence of the unequal distribution of annual precipitation (Arenillas 2000); these climatic conditions forced the construction of reservoir dams. In fact, four of these dams were constructed so as to ensure the water supply to the Roman towns

Tabla 1. Principal dams of the Roman era in Spain

NAME	HEIGHT	RIVER	BASIN
Almonacid de la Cuba	34,0	Aguasvivas	Ebro
Proserpina	21,6	Las Pardillas	Guadiana
Cornalvo	20,8	Albarregas	Guadiana
Ermita Virgen del Pilar	16,7	Santa María (Aguasvivas)	Ebro
Alcantarilla	15 to 20	Guajaraz	Tajo
Muel	13,0	Huerva	Ebro
Pared de los Moros	8,4	Farlán (Aguasvivas)	Ebro

mentioned above: Muel to *Caesaraugusta*, Proserpina and Cornalvo to *Augusta Emerita*, and Alcantarilla to *Toletum*.

However, this system was not the standard pattern applied by the Romans to resolve supply problems. In reality they only used it when, with good reason, climatic conditions forced them to do so.[3] In most cases (and frequently in Spain) they opted for riverhead collections by means of diversion weirs or intakes direct from sources.[4]

Nevertheless, when the Romans built regulation dams in Spain they frequently differed from this pattern. Of the three areas aforementioned this can be found in the basin of the Ebro (area of *Caesaraugusta*) where large dams were located, as a rule, in the middle stretch of rivers of some importance. On the other hand, in the mid-west of the Peninsula (*Emerita* and *Toletum*) these works were always situated on riverheads or streams with small catchment basins.

These differences of criteria regarding the emplacements can also be found in the structural solutions adopted: in the Ebro the highest dams are masonry dams, whilst in the Tajo and Guadiana the dams are earth dams with retaining walls upstream. However, the smallest dams —where other forms may be found— form a more homogeneous group in general.

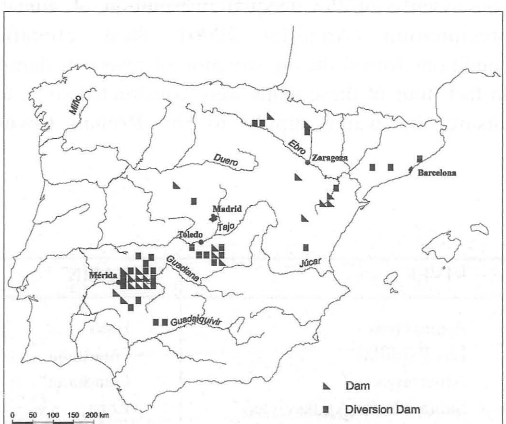

Figure 1
Location of the dams in the Roman era

Figure 2
Supply system to *Toletum* (Alcantarilla dam)

STRUCTURAL FORMS

In the majority of the dams built in Spain by the Romans there is a basic characteristic construction element, almost systematically repeated: the retaining wall, used to achieve watertightness of the structure. Other elements, though not always, were added to ensure or complete the stability of the system. The Roman retaining wall is a very simple concept: a lime concrete core (*opus caementicium*), framed by two wall sections made of masonry (*opus incaertum*) or ashlar (*opus quadratum*). When the masonry was of poor quality other wall sections were attached to the

first ones being of increasingly better quality towards the exterior. The most important element of this system was the core of *opus caementicium*, whose purpose was to comply with the objective of retaining the water.

The large dams on the Ebro basin

The dam which probably conforms best to the strict pattern of retaining wall is the one known as La Pared de los Moros (The Moors' Wall).[5] It is located near Muniesa (Teruel) in a secondary waterway, the Arroyo Farlán, the rightward tributary of the river Aguasvivas,[6] which at the same time is a branch of the Ebro, also on the right bank (Arenillas, Díaz-Guerra y Cortés 1996). The dam initially formed a reservoir of approximately 150,000 cubic metres capacity; nowadays it has a breach in its middle section. The characteristics of the masonry —not properly laid down in general, and the layout— somewhat winding (as with the layout of the limestone outcrops in the area) enable us to think of it as a later Roman work of rural style, perhaps dedicated, at least in part, to irrigation. The structure is nearly 8,5 metres in height and has a crest length of around seventy metres. Its form is as previously indicated: a single wall of nearly three metres thickness, formed by two masonry wall sections laid with lime mortar (*opus incaertum*) and a core of *opus caementicium*. The coverings are of 1,10 metres thickness each and are built with local limestone, lightly worked. The core reaches up to seventy centimetres thickness.

The basic fault of this structure is its extreme thinness[7]. With such a risky geometry the presence of an earth embankment downstream should be expected, but the materials existing there have not permitted the detection of the remains of such a complementary structure. As a result, the Pared de los Moros undoubtedly broke, and probably quite early as the sediments of the reservoir have not developed much, although they also could have been swept away by the waters after the dam breached. They can be observed, in particular, on the right bank where they show up without excessive re-workings —natural or artificial— since their deposition.

The calcareous concretions observed in the downstream face of the dam are not abundant, which could indicate that the retaining wall worked properly from the point of view of impermeability. In fact, the *opus caementicium* forming its core is of good quality.

The best pattern of a dam formed by a reinforced retaining wall (that is, an improved version of the previous pattern in terms of resistance) is the dam of Almonacid de la Cuba[8]. This is the highest dam from the Roman era preserved in the world (thirty-four metres). It is located on the river Aguasvivas and has a catchment basin of about 1.000 square kilometres. Built in the era of Augustus and rebuilt and repaired several times, this dam has a peculiar feature which makes it even more interesting: the preserved structure is an important reconstruction of a previous structure of completely different form.

The first dam raised on the closure site of Almonacid must have been formed by three arches,

Figure 3
La Pared de los Moros

Figure 4
Dam of Almonacid de la Cuba

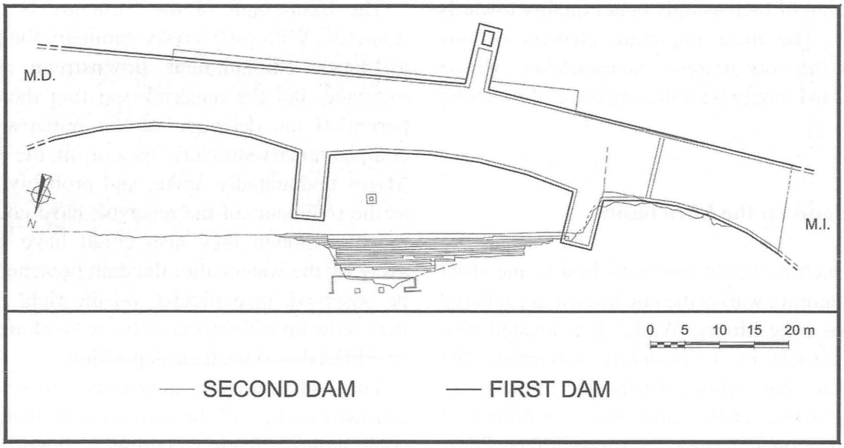

Figure 5
Layout (assumingly) of the early dam of Almonacid

one central and two side ones leaning against two large buttresses (Arenillas, Díaz-Guerra y Cortés 1996). This first dam must have been breached quite early, even perhaps in the later phases of construction and it also must have been rebuilt at once, its original structure being substantially modified, becoming the typical straight gravity dam. The breach of the dam was certainly partial and probably was located on an isolated point, the central arch for example, as in the latter reconstruction many of the original elements were entirely or partly preserved: the arch of the left edge, with elements from the buttress it was leaning against, or the intake tower, among others (Arenillas, Díaz-Guerra y Cortés 1996; Hereza et al 1996).

The first dam of Almonacid has been dated by the C14 method applied to two wooden samples obtained in a drilling. The age calibrated for those samples dates the construction of this work to the era of Augustus and, particularly, in the early years of the first century A.D. Therefore the second dam belongs most certainly to the first decades of the same century and, perhaps even to the very era of Augustus (Arenillas, Díaz-Guerra y Cortés 1996).

The definitive dam of Almonacid is a retaining wall, highly reinforced in its main part, with a thinner, short block on the left edge, where the weir is. The main part of the structure —very robust— encloses the deepest area of the valley and consists in section of a rectangular central body and two stepped faces; downstream the stepping is double. In the central body a retaining wall stands out which, according to the data obtained from the drillings, reaches between 10 and 12 metres thickness, of which the 2,70 central metres belong to a lime concrete core (*opus caementicium*). This core is framed between two double masonry wall sections (*opus incaertum*) with an average thickness of about 3,70 metres upstream and 4,60 metres downstream. In both cases the masonry located beside the core is of worse quality than the exterior ones.

The retaining wall belongs to the first dam and perhaps then had an ashlar facing (*opus quadratum*), as may be deduced from the samples obtained in some drillings. This wall was considerably reinforced on reconstruction: a masonry wall of about 9 metres thickness was built downstream, covered on the face by a wall section of *opus vittatum* (limestone pieces placed in horizontal courses) where a large stepped-in skirt was attached, the lower of the two preserved on that side. The reinforcement would be increased later on with two new stepped-in skirts, one on each side.

After this major reconstruction and as a consequence of the repeated effects of the floods on the river Aguasvivas, the dam had to be continuously repaired. In the preserved masonry various reconstructions can be observed and from the study of

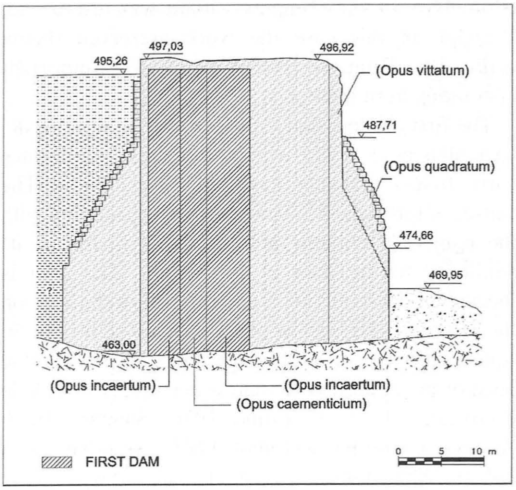

Figure 6
Dam of Almonacid. Section

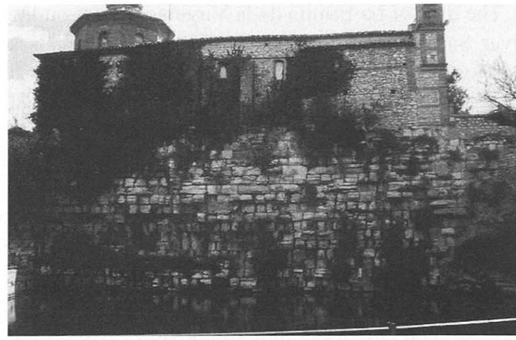

Figure 7
Dam of Muel

the reservoir deposits a period of abandonment during the second half of the first century has been detected (Hereza et al 1996). The most important works can be dated to the era of Claudius (41–68) and Trajan (98–117) (Arenillas, Díaz-Guerra y Cortés 1996). In the latter period the dam was heightened in order to alleviate silting effects, which must have been significant (Hereza et al 1996).

Thanks to these measures and the later silting of the reservoir, the dam has been preserved to date after some medieval and later works by which time the dam had already become a diversion dam. It still complies with this function, diverting the waters through the former Roman canal up to the irrigation area of Belchite, located approximately 8 kilometres downstream.

The second structure designed by the Romans in the straight of Almonacid may definitely be considered as valid —although excessive by present criteria— from a resistance point of view. The almost 40 metres thick foundations as opposed to 34 metres maximum height assured this condition. And yet in any case the Romans did not deal properly with two important matters: firstly the weir, with an obvious lack of capacity to cope with the main floods.[9] Secondly the poor quality of the lime used to build the retaining wall core. The first of these faults may explain the cause of the major reconstruction of the first dam and the various later repairs. The second has been checked according to the samples obtained from the drillings, where in some cases the lime of the *opus caementicium* core of the retaining wall is observed to be unset at many points. This fact certainly led to the lack of general impermeability of this fundamental building element, as may be deduced from the many calcareous concretions appearing on the downstream face of the dam, particularly on the wall section of *opus vittatum.*

Two other important dams on the basin of the Ebro are Muel and La Ermita de la Virgen del Pilar. The first is of great interest as it belongs to one of the three (or four) Roman supply systems to *Caesaraugusta.* However it has not been researched in detail since it is not an easy task due to the fact that the reservoir which it formed on the river Huerva (branch of the Ebro on the right bank) is completely silted up. Nevertheless, the facing masonry downstream can be seen at a height of about 13 metres. This masonry is made of dressed ashlar with courses about 50 cm in height. The majority of the ashlars are laid in stretcher courses, though some are placed in header courses. The masonry thickness has been sized at around 7 metres (Castillo 2002) although it could be thicker in the lower part of the work. Its internal structure is not known and, although there are filtrations through the face, it seems reasonable to think of some waterproof element inside (a core of *opus caementicium* for example). The dam could have been easily built in the era of Augustus, as it is related to the water supply to *Caesaraugusta*, and perhaps at an early date, for the facing masonry would fit this period.

The dam of La Ermita de la Virgen del Pilar on the river Santa María, tributary of the Aguasvivas, is a gravity dam formed by a complex retaining wall built in two stages. Only the part of the structure located at the highest point of the left edge is preserved, where a heterogeneous succession of masonry can be observed, allowing the explanation of two-stage construction and also the final heightening of the structure. However the total thickness of the dam is only 6,90 metres, which for its maximum height of 16,60 metres shows a clear situation of instability. In fact the dam collapsed, though not very early, as the reservoir sediments grew quite thick, as may be observed on the slope upstream from the dam. According to its structure it could have been built following the model of Almonacid, although the masonry quality is remarkably poorer. In any case the height of this work is surprising (taking into account the Roman standards) as is its function, as no reasonable destination has been traced for its waters (Confederación Hidrográfica del Ebro —Ingeniería 75 2000).

Figure 8
Dam of la Ermita de la Virgen del Pilar

The large dams on the basins of the Tajo and Guadiana

Also in the large earth dams in the mid-west of the peninsula the retaining wall was used as a fundamental element to retain the waters. In two cases (Alcantarilla and Proserpina) this solution was applied with very strict design criteria; as for the other (Cornalvo) a more complex variant was resorted to, although in this case the work preserved (being originally Roman) possibly shows important alterations from later eras.

The first of these dams must be that of Alcantarilla (Arenillas et al 1999), which has been in ruins since early times, probably since the Roman era. The causes of this breach have been analysed starting with the numerous remains preserved and thanks to its similarity to the dam of Proserpina. The dam is located on the river Guajaraz, tributary of the Tajo on the left, on a high level of its course (with only 50 square kilometres of drainage area) and it was the head of the important Roman water supply to Toledo (Fernández 1961; Celestino 1976; Sánchez 1977; Aranda, Carrobles e Isabel 1997; Arenillas et al 1999). The dam is formed by a large earth embankment (highly degraded today) and by a retaining wall upstream, of which some traces are preserved almost intact and various blocks strewn over the ground. The maximum length of the dam must have exceeded 800 metres and its maximum height may be estimated as between 15 and 20 metres. The embankment is formed by sandy clays typical of the altered granites of the basin and hence, is unlikely to be highly impermeable; consequently the retaining wall again carries out the function of avoiding the passage of the reservoir waters, leaving the resistance action to the embankment as the retaining wall is very thin for its height, about 4 metres thick at its base.

As observed in some blocks, the retaining wall is formed by a lime concrete core (*opus caementicium*) of about 60 centimetres thickness, manufactured with small pebbles (*caementa*), 5 cm maximum, and a great deal of aggregate. The core is situated between two masonry wall sections (*opus incaertum*) of variable thickness, oscillating between 90 cm and 1,50 metres. The upstream wall section must have been composed wholly of ashlar stretcher courses (*opus quadratum*) of which some course traces are preserved in the block that stands on the left edge. They are fine worked pieces of about 50 cm height, 60 cm thickness and lengths reaching over one metre. The downstream face follows the vertical line, whilst the upstream face is slightly separated from it.

The main problem —well known nowadays— of a dam of the above characteristics is its instability at empty reservoir: a retaining wall as thin as the one of

Figure 9
Dam of Alcantarilla

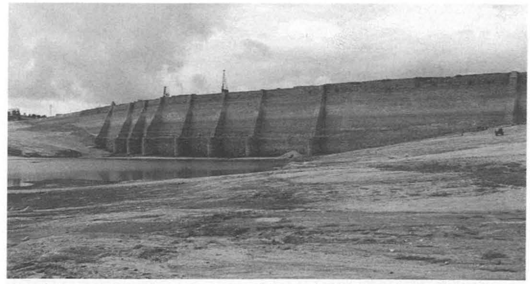

Figure 10
Dam of Proserpina upstream

Alcantarilla hardly resists the embankment push in those circumstances and least of all when it is saturated, a situation which may arise from filtrations through the retaining wall or from overspills on the crest. In fact the dam was breached due to the embankment push, as the retaining wall is strewn towards the reservoir in the ruined middle portion, although some elements show up downstream; the position of the latter can be explained by movements during flood episodes after the breach. Nevertheless, the dam is likely to have breached during a flood, as most certainly (along with Cornalvo and Proserpina) it was not provided with a weir.

In the dam of Alcantarilla there are still remains of two intake towers, one on the lowest point of the closure and the other located on the right bank and therefore, at a higher position than the previous; both were attached to the retaining wall downstream. In the central tower the intake must also have functioned as a dewatering outlet, as the whole reservoir could not be emptied from the other tower; it is the same pattern found in Proserpina.

In summary, the form adopted by the Romans in the dam of Alcantarilla was, in principal, correct but they did not count on two important factors: the floods of the Guajaraz and the lack of resistance of the retaining wall to the embankment push at empty reservoir. In Proserpina, whose structure follows the same form, some improvements were made; this happened also in Cornalvo. Hence, Alcantarilla is likely to be the most ancient of the three large Roman dams preserved in the mid-west of the Peninsula (Arenillas et al 1999).

The dam of Proserpina is a much better known work than the above as it is still working (although dedicated to aims other than those intended by the Romans) and has recently been studied (Arenillas, Martín y Alcaraz, 1992; Alcaraz et al, 1993; Confederación Hidrográfica del Guadiana-Ingeniería 75, 1996; Martín et al, 1998). It is located on the course of the brook of Las Pardillas, a sub-tributary of the Guadiana on the right bank..

In 1991 the Confederación Hidrográfica del Guadiana (Water Management Administration) started a series of activities for the refurbishment of the dam and the regeneration of the reservoir, whose waters had reached a high degree of eutrophication and could not be drained, as the deepest outlets —the original Roman intakes— had lost their function due to the partial silting up of the reservoir. The removal of these materials revealed nearly seven metres of masonry whose morphology contrasts to some degree with that of the upper part of the structure, the one known up to that date. This activity and the data obtained from several drillings and other investigations carried out, enabled a good evaluation of the structure.

The dam of Proserpina is formed by a masonry wall (the retaining wall) to which an earth embankment is attached downstream. The retaining wall is formed by two granite masonry wall sections —ashlar, banded stone or masonry, depending on the areas— with a core of lime concrete between them. The maximum height of this wall is of 21,60 metres of which the lower 6,60 metres belong to the recently discovered masonry. In layout the dam follows three straight alignments with a total crest length of 427,80 metres. On the left edge there is also an auxiliary wall

Figure11
Dam of Proserpina. Detail of a buttress

of about 100 metres length used to enclose some areas where the ground remains below the crest of the dam.

The upstream face of the retaining wall is vertical in the lower 6,60 metres and inclined in the rest, which can be achieved by the stepping of successive ashlar courses forming it in that area. Nine buttresses emerge from this face, distributed irregularly along the central sector of the dam; eight of them have their origin in the lower masonry. These eight buttresses are vertical in the part belonging to the oldest masonry and from that area they extend up to the crest with a gentler slope than that of the wall, achieved similarly by offsetting the successive courses. In the lower section these buttresses finish with a semicircular section at about 4,5 metres from the face; in the upper stepped area all the nine buttresses are of rectangular section.

The downstream face remains covered by the earth embankment almost up to the crest. Nevertheless its verticality has been proved by means of drillings and scrapings in several points and probably is a general characteristic throughout the structure. The indicated probing permitted the detection of sixteen buttresses in the middle section of the dam. They are vertical masonry elements of approximately 1,40 metres width and three metres length, split about six metres between each axis. All the buttresses finish two metres below the crest, just where a 30 cm ledge is located, which shows up along the face of the dam. The horizontal drillings made in the retaining wall have indicated a foundation thickness of 5,90 metres.

The reservoir intakes are placed in two towers attached to the retaining wall in its downstream section, therefore embedded in the earth embankment, and emerging at a height so as to allow access. The main tower is located on the deepest part of the closure site and has an irregular section, although almost square, of about 5 or 6 metres on its exterior sides. This tower contains two intake series. The lower (of the Roman era) is formed by two lead pipes of about 22 cm interior diameter, placed at more than three metres over the foundation level. Nearly four metres higher up there is another intake cut into a granite flagstone which probably belongs to works from the seventeenth century. The other tower is located on the left bank at about ten metres over the river course. It also has a slightly square section of about 7 metres on the exterior sides. This tower contains an upper intake, located nearly twelve metres above the lower one. Until the 1940's the Roman pipe must have been preserved, being later replaced by the cast pipe presently in existence.

This intake is particularly interesting: it is the only one which by level allows the transfer of water from the reservoir to Mérida across the bridge of Los Milagros as the level of the conduit above this aqueduct is higher than the level of the other Roman intake. This fact assures the Roman character of all of the dam[10], although this does not exclude the subsequent repair or reconstruction of the upper section of the structure. On the other hand, some absolute dates are provided for the dam of Proserpina from two wood samples obtained from the lower part of the masonry by means of a horizontal drilling. Analysed by the C14 method they enable the dating of the construction of this masonry to the era of

Figure 12
Dam of Proserpina. Roman intakes

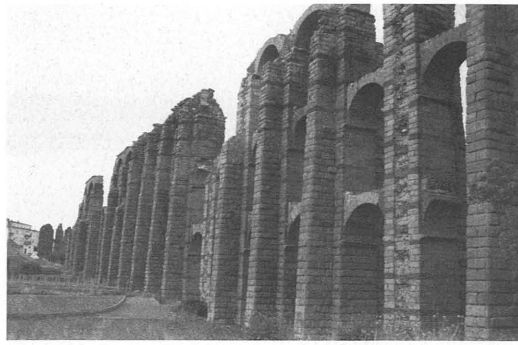

Figure 13
Aqueduct of Los Milagros

Trajan (98–117) (Confederación Hidrográfica del Guadiana-Ingeniería 75 1996). Therefore it seems reasonable that the construction of the aqueduct of Los Milagros should be dated to the same period or even somewhat later.

The dam of Proserpina, with a similar structure to that of Alcantarilla (although reinforced with buttresses) has outlived the latter almost two thousand years. It is not clear, however, that such measures have played a part in the longevity of the structure, since the upstream buttresses (probably built in order to improve the resistance of the retaining wall against the push of the embankment at empty reservoir) do not seem to be too effective as a result of the distances between them.[11] The reason for the stability of the retaining wall must be basically the low probability of important floods on the small stream feeding the reservoir (with a basin of 8,5 square kilometres) even adding the effects of the contiguous basin, from which flows were transferred to the reservoir (another 15 square kilometres)[12]. This practice ensures greater flows in normal circumstances, but at the same time enables their elimination by stopping the transfer under extreme circumstances. This is surely the reason why the dams of Cornalvo, Proserpina and Alcantarilla were not provided with weirs,[13] for the outlets must have been considered sufficient to handle the respective reservoirs. This assumption turned out to be valid in Proserpina and Cornalvo but not in Alcantarilla, where the catchment basin is somewhat larger.

The Roman dam of Cornalvo is located on the river Albarregas, tributary of the Guadiana on the right bank, about fifteen kilometres from Mérida. It was built in order to improve the previous exploitation of the water supply to *Emerita*, which had its origin in a series of collection galleries tunnelled into the deposits of the river Albarregas, in the area later flooded by the reservoir.(Martín et all 2000). These galleries converged at one point (Macías 1929) where the conduit towards Mérida started. The dam must have been built when the water from this source proved to be insufficient for the town; then an intake tower was raised at the spot where the former galleries met, near the dam but inside the reservoir. Therefore the Cornalvo intake tower turns out to be a unique element in the dams from the Roman era built in Spain.

The dam of Cornalvo is not yet properly researched, but it mainly follows the pattern of Alcantarilla and Proserpina: a large embankment sheltered upstream by a structure element —not exactly a retaining wall— which carried out the function of preventing the passage of water. From the data available today it seems that this structure is formed by three longitudinal walls (parallel to the direction of the dam), another series perpendicular to the latter and all of them covered by the face upstream of the dam, which has a gently rising slope. The enclosures formed by this group of walls are filled with materials of different types.

It is not clear whether this system was adopted by the Romans, for it would be quite an innovative pattern for that era, at least in Spain. It is possible though, that the Romans just built a wall alongside the embankment —perhaps a retaining wall in this

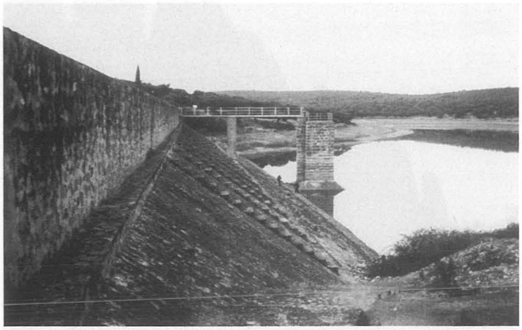

Figure 14
Dam of Cornalvo

case— and that they reinforced it somehow, questions that must be answered once the structure has been fully researched. It is known that the dam has been repaired in several occasions and hence, it is possible that part of this structure belongs to some of these activities.[14]

Small Dams

On small works (Appendix) the Romans quite frequently maintained the forms followed for the large dams, but in many occasions they simplified these structures and even adopted different ones. For example, the standard retaining wall was replaced in many cases by a simple wall of *opus incaertum*. Therefore the water leakage would surely be greater than that resulting from the masonry of *opus caementicium*, but obviously the problem could be acceptable for most of the diversion weirs, as well as low height dams. There is one case (dam of El Paredón) and perhaps more, where the Romans tried to solve this problem by adding a mortar rendering to the upstream face. In this dam the rendering is, essentially, an *opus signinum* like the one used by the Romans to dress and water-proof canals and tanks (Castillo 2002). On small dams the pattern of an earth dam with retaining wall upstream is also used, in some cases with the required buttresses upstream, as can be observed in Las Tomas (Guadiana) or El Paerón I (Tajo). A very common solution in these structures also is the buttress dam, formed by a retaining wall, a simple wall or multiple arches, leaning against the buttresses, located downstream. The best example of this type may be, due to its importance (over 600 metres length, although only 4,80 metres height) the dam of Consuegra, on the basin of the Guadiana. It had a retaining wall upstream, numerous buttresses and perhaps an embankment downstream, of which no remains are left. (Castillo 2002). Similar to this dam but with no embankment is the dam of Araya, and with multiple arches the dam of Esparragalejo, both near Mérida. On the Ebro basin the dam of Villafranca (150 metres length and a reduced height of 3 metres) is the most notable of this type.

An original form, as we only have one example, is the gravity arch dam. To this type belongs the dam of Puy Foradado in the important hydraulic system of Los Bañales (Ebro basin). It is a circular structure, with approximately 56 metres of development and reduced height (about 2 metres) used as diversion weir in the mentioned system. The upstream face is formed by four ashlar courses; it is the only visible masonry nowadays, since the reservoir is completely silted-up (Castillo 2002).

One last dam also to bear in mind, for its structure is somewhat peculiar, is the dam of Iturranduz, at the head of the Roman water supply to the town of Andelos (Ebro basin). It is a double dam, or rather duplicated, as two structures have been preserved, one probably from the second century, the other posterior (third or fourth century). The eldest is located downstream from the other and it was a wall of over 100 metres length, nearly one metre thick and a little more than four metres in height (as per the remains

Figure 15
Dam of Consuegra

preserved) leaning downstream against nine square section buttresses with 2,50 metres side length. All the masonry is made of lime concrete (*opus caementicium*) and the traces of the wooden formworks used for its construction can be observed in it. The second structure is a simple wall with buttresses too, but in this case such elements were located upstream. The length of this wall is greater than the previous one (about 150 metres) and the thickness is less (65 cm); its height is not easy to estimate, but it could not have exceeded the other structure. The wall leans against an uncertain number of buttresses, which, according to the remains could be more than fifteen. In this case the masonry is bedded with lime mortar and laid in courses (perhaps, *opus vittatum*). By its position on the ground this second structure must have been designed as a reinforcement or repair of the first, as the space between them must have been filled with earth (which was extracted when the area was being excavated). Some remains of an intake tower are preserved in this second dam, where the conduit towards Andelos must have started.

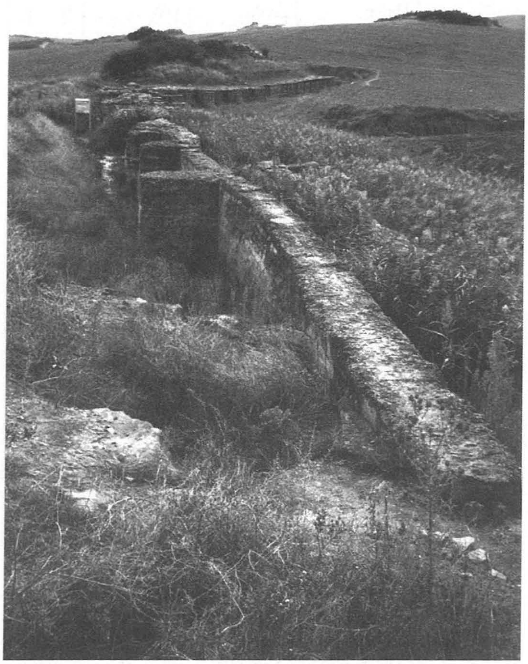

Figure 16
Dam of Iturranduz (inferior)

Auxiliary elements of the dams

The lack of weirs is one of the characteristics —anomalous we would say nowadays— of nearly all the Roman dams located in Spain. Only in Almonacid may work of this type be clearly identified. However, and as stated above, its capacity was very low and therefore, hardly effective. It is also true, as far as we know today, that the Romans never built dams on large plentiful rivers and most of the time they simply intercepted minor streams. Accordingly, it is possible that the Romans really intended, in those cases, the formation of large deposits at the heads of the hydraulic systems they built (*caput acquae*). In this manner they could control the reservoirs on low flow watercourses and during small floods by simply using the outlets installed in the dams. But in Spain, in rivers like Aguasvivas (Almonacid), Huerva (Muel), Guajaraz (Alcantarilla) or even Santa María (Virgen del Pilar), circumstances were certainly distinct; despite none of these rivers being especially plentiful (although their floods can be considerable). However, not even in those cases did the Roman tackle the problem adequately. In some masonry dams the floods did not manage to cause ruin to the structure (second dam of Almonacid or Muel) but, logically, the same cannot be said of the earth dams (Alcantarilla). Probably due to this fact the latter form was only repeated in other dams located in areas where the probability of large floods was very low (Cornalvo and Proserpina, among the largest works).

Other interesting elements are the intake towers built by the Romans, systematically as it seems on the large dams, but also on smaller ones. In all known cases, except in Cornalvo, these works were attached, upstream or downstream, to the masonry of the structures, with access from these or from the embankments to the chambers where the opening and closing elements of the conduits were located (almost always bronze pieces on lead pipes). The breakage or breakdown of these elements must have caused complicated problems; as such situations should lead systematically to the flooding of the tower by the reservoir water. In Proserpina, when the sediments that had partially filled the reservoir had been removed, a large wooden plug was found (dated to the Roman era by C14) that must have been used to close the conduit from the reservoir in this kind of event. In these cases the problem must have been the removal of the plug under a full reservoir.

APPENDIX. Reservoir dams and diversion weirs from the Roman era in Spain

NAME	DIMENSIONS			SITUATION			TYPE	DATE CONST.
	L	T	H	RIVER	BASIN	PROVINCE		
RESERVOIR DAMS								
Almonacid de la Cuba	120,0	38,0	34,0	Aguasvivas	Ebro	Zaragoza	RRW	I
Proserpina	427,8	5,9	21,6	Las Pardillas	Guadiana	Badajoz	E (RW)	I–II
Cornalvo	194,0	26,0	28,0	Albarregas	Guadiana	Badajoz	E (RW)	I–II
Alcantarilla	>800,0	4,0 (?)	20,0	Guajaraz	Tajo	Tolero	E (RW)	I
Ermita de la V. del Pilar	80,0	6,9	16,6	Sta. María	Ebro	Teruel	RRW	I–II
Muel	60,0	7,0 (?)	13,0	Huerva	Ebro	Zaragoza	RRW	I
La Pared de los Moros	68,0	2,7	8,4	Farlán	Ebro	Teruel	RW	III
Esparragalejo	320,0	2,2	5,6	Albucia	Guadiana	Badajoz	B (RW)	I
Consuegra	>632,0	2,6	4,8	Amarguillo	Tajo	Toledo	B (W)	III–IV
Las Tomas	95,0	1,9	5,2	—	Guadiana	Badajoz	B (W)	IV
Iturranduz o Andelos inf.	102,0	1,0	>4,0	San Pedro	Ebro	Navarra	B (W)	II–III/IV
Iturranduz o Andelos sup.	150,0	0,7	(?)				B (W)	
Arévalo	50,0	3,0	6,0 (?)	Arevalillo	Duero	Ávila	DW	II
El Paredón	141,1	2,7	4,5	Paredón	Guadiana	Badajoz	E (RW-B)	III
La Pesquera	100,0	5,6	4,0	—	Ebro	Zaragoza	W	?
Araya	139,0	1,8	3,7	—	Guadiana	Badajoz	B (RW)	II
Vega de Sta. María	97,8	3,5	3,6	Heras	Guadiana	Badajoz	B (RW)	?
Villafranca	150,0	2,2	3,0	Jiloca	Ebro	Teruel	B (RW)	II–III
Paerón I	80,0	1,2	2,4	Sta. María	Tajo	Toledo	E (W-B)	I–II
Los Paredones	80,0	2,5	>2,0	Gitano	Guadiana	Badajoz	W	I–II
El Peral	30,0 (?)	1,0 (?)	2,2 (?)	Norias	Guadiana	Badajoz	W	I–II
La Cuba	52,0–180 (?)	0,8	>2,0	Cuba	Guadiana	Badajoz	E (W)	II–III
DIVERSION WEIRS								
Río Frío	13,4	0,7	1,1	Aceveda	Duero	Segovia	W	I
Pont d'Armentera	35,0	0,8	1,5	Gayá	Tarragona	Ebro	W	II–IV
Azud de los Moros	40,0	0,7	0,9	Tuéjar	Turia	Valencia	W	I
Arroyo Bejarano	40,0	2,0	3,5	Bejarano	Guadalquivir	Córdoba	W	I
Palomera Baja	15,0	1,0	2,2	Palomera	Guadalquivir	Córdoba	W	III
Puy Foradado	56,0	1,0	2,0	—	Ebro	Zaragoza	A (W)	II–III

APPENDIX. Reservoir dams and diversion weirs from the Roman era in Spain (continuación)

NAME	DIMENSIONS			SITUATION			TYPE	DATE CONST.
	L	T	H	RIVER	BASIN	PROVINCE		
Pineda o Ca'La Verda	25,0	1,5	2,5	Pineda	Ebro	Barcelona	RRW	III
Las Adelfas	(?)	(?)	(?)	Las Adelfas	Guadiana	Badajoz	W	II
Las Muelas	200,0	3,4	3,0	Las Muelas	Guadiana	Badajoz	B (RW)	II
Cañada del Huevo	100,0	5,0	2,5	—	Guadiana	Badajoz	B (RW)	II
Las Mezquitas	(?)	(?)	1,6	—	Guadiana	Badajoz	RRW	II
S. Martín de la Montiña	(?)	(?)	3,0 (?)	San Martín	Tajo	Toledo	W	I–II
Odrón y Linares	(?)	(?)	(?)	Odrón-Linares	Ebro	Navarra	W	?
Arroyo Salado	50,0	2,0	7,0	Salado	Ebro	Navarra	RW	?
Melque VI	19,5	2,5	4,5	Las Cuevas	Tajo	Toledo	RW	?
Charca de Valverde	170,0	3,0	>3,0	La Charca	Guadiana	Badajoz	E (RW-B)	?
Azud de la Rechuela	29,0	3,0 (?)	3,0	Aguasvivas	Ebro	Zaragoza	B (RW)	?
Les Parets Antiques	30,0	2,3	3,0	Riera S..Sebastián	Ebro	Barcelona	W	III–IV
Mesa de Valhermoso	98,0	1,8	3,0	Valhermoso	Tajo	Toledo	E (RW-B)	II–III
Castillo Bayuela	30,0	1,5	3,0	Guadamera	Tajo	Toledo	B (RW)	II–III
Moracantá	40,8	1,9	2,1	Guazalote	Tajo	Toledo	RW	I–II
El Hinojal (Las Tiendas)	230,0	1,6	1,3	Rto. Charcoblanco	Guadiana	Badajoz	B (RW)	III–IV
Paerón II	30,0	1,1	>1,5	Sta. María	Tajo	Toledo	B (RW)	I–II
El Argamasón	14,7	1,4	1,3	Tripero	Guadiana	Badajoz	RW	II–III
Balsa de Cañaveral	30,0	2,4	1,2	—	Tajo	Cáceres	E (W)	IV
El Peral II	7,6	0,7	>0,9	Las Norias	Guadiana	Badajoz	B (RW)	?
Valencia Ventoso	60,0–80,0	1,6	>0,8	—	Guadiana	Badajoz	W	III–IV
El Chaparral	50,0	1,1	>0,8	La Alcazaba	Guadiana	Badajoz	W	III–IV
Monroy	(?)	(?)	(?)	—	Tajo	Cáceres	W	?

RW: retaining wall; RRW: reinforced retaining wall; W: simple wall; DW: reinforced wall; E: earth dam; B: buttress dam; A: arch dam
L: crest length; T: thickness(in earth dams refers to thickness of the wall) H: maximum height

A tower of unique form, already referred to, is that of Cornalvo, located inside the reservoir. The operations performed from it are not easy to understand (with the means the Romans had to hand). This is why, most probably, it was a decorative element that may have been used to protect the beginning of the conduit, for the opening and closing operations could be done from inside the dam or immediately downstream.

NOTES

1. This is a doctoral Thesis, undertaken by the second undersigned and co-directed by the first in cooperation with Professor F.Santos (Castillo 2002).
2. Among these dams the diversion weirs of mining character have not been accounted for, though they are particularly numerous in the former Roman prospects —and previous ones— of the north-west of the peninsula, dedicated to gold extraction.
3. According to the data available, the large regulation dams built by the Romans are exclusively located in the Mediterranean areas poorly favoured with precipitation: Southern France, Hispania, North Africa and Middle East (Cf. Schnitter 1994).
4. In the Baetica province alone of the Spanish Peninsula 26 towns with water supply from the Roman era have been mentioned, and in this environment no important regulation dam seems to have been built since (Castillo 2002).
5. Up to recent times in Spain all *the ancient*, with no established or approximate date was said to be «from the Moors», that is, from the period of Muslim occupation. Lately, with greater levels of information and many more «experts» giving opinions over ordinary people, almost all *the ancient* has turned out to be «from the Romans».
6. In the basin of the river Aguasvivas there is a remarkable accumulation of dams and weirs, whose construction extends from the Roman era (with three — or four— structures) until the twentieth century and counts some interesting examples from the fourteenth to sixteenth centuries. (Arenillas, Díaz-Guerra y Cortés 1996).
7. For a rectangular section structure and average specific weight of 2 t/m3 (which must be equivalent to that of the Roman masonry), the strict tilting stability is achieved with a height double the thickness considering, logically, the effect of uplift, which very probably the Romans did not know how to value, in spite of Archimedes.
8. The date of construction of this dam, which had not been entirely researched until the 1990's (Arenillas, Díaz-Guerra y Cortés 1996), has been assigned many times to the Muslim era and even, in more detail, to the reign of Jaime I of Aragón (thirteenth century), according to the tradition of the eighteenth and nineteenth centuries. And this despite the fact that Ponz ([1788] 1989) had already written that «It seems to be from Roman Times but is attributed to King Jaime I who may have repaired it» and Galiay (1946) who later reiterated its Roman origin. Even Norman Smith, some years after Galiay, insists on placing the dam in the era of Jaime I in an extensive work concerning ancient Spanish dams that unfortunately contains some inexplicable errors.
9. In the aforementioned doctoral thesis (Castillo 2002) the calculation of these effects is included and it is demonstrated that the weir only has the capacity to clear less than twenty year return period floods. That is to say, during the first century A.D. (from Augustus to Trajan) and later, of course, the reservoir must have spilled over the crest on numerous occasions with the consequent erosion at the dam foot
10. The Proserpina dam has always been considered Roman since the time of first investigations. However, when the lower section of the work was discovered there were some opinions — not written, as far as is known — which began to doubt the Roman origins of the upper section of the structure.
11. As pointed out by C. Fernández Casado (1961). In addition, recent calculations show that the retaining wall would not be stable at empty reservoir and saturated embankment (Castillo 2002).
12. In Proserpina, Alcantarilla and Cornalvo diversion weirs were built on the courses of adjacent basins and from them feeding conduits to the respective reservoirs.
13. Some references to works of this type in some of these reservoirs are modern or correspond to natural erosion.
14. There is some data concerning an important repair work carried out in the eighteenth century, when the Conde de Campomanes refurbished the work with the purpose of using the water of the reservoir in a paper factory constructed some kilometres downstream from the dam. The structure which is preser
ved today may correspond in part to this era, as at the time the large dam of El Gasco close to Madrid was constructed with a cell structure which to a certain degree is similar to that of Cornalvo (Martin et al 2000).

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Techniques, designs and terminology of elevation of stonework bridges during modern age

Begoña Arrúe Ugarte

The analysis of handwritten sources related to the construction of bridges during the Modern Age, which began during the elaboration of the catalogue of bridges in La Rioja (Arrúe and Moya, coord., 1998), let me present to the First National Congress of Construction History (Arrúe 1996), a preliminary study on the system of foundation under the water used in their building. The paper I present today aims at completing the findings those sources add to the knowledge of implementation of elevation of bridges and at establishing some conclusions about the models, techniques and terminology generalized during that period in the context of Castilla, connecting them with the ones defined in the contemporary construction of bridges in other Spanish regions and in France, the nearest references, as well as to the ones proposed by the theorists of architecture.

Legalized written documents of contracts for the building of a bridge go with full constructive conditions which, very often, focus upon the types of foundation to follow. Their relevance for the survival of the building is shown in the presentation of the layouts or detailed plans on the grillage or pile foundation must be carried out, especially in repairing projects. This is the case in, for example, Manuel del Olmo's for the bridge of Viveros upon the Jarama, Madrid, around 1686 (Corella 1992, 166), or Juan Martínez's and Martín de Urtázar's 1694 for the repairs of the bridge in Nájera upon the Najerilla along St. Jacques Way through La Rioja (Arrúe and Moya, coord., 1998, 1:423). But, at the present time, we are interested in highlighting the methods used for the elevation of the fabric after the conclusion of the works of foundation and the previous disposition of caissons, in the way they are documented in the projects of the work contracts.

The foundation done, they go ahead with the building of piers. The use of this term was not common until the XIX c. being more general in the revised documental sources from the XVI to XVIII c. the terms *cepa* or *machón*. The term *pilar* was also used preferably along the XVI and the first half of XVII c.[1]. The author of Tratado de Arquitectura, preserved in the National Library, in the middle of XVI c., mentions *pilares* (Anonymous, c 119, 225), and also Simón García in *Compendio de architectvra y simetría de los templos . . .*, 1681, which copies part of the treatise written by Rodrigo Gil de Hontañón in the XVI c. and, also, the *Arte y Uso de Architectura*, by fray Lorenzo de San Nicolás, 1639 and 1664, who uses *cepa* (Huerta 2000, 521). On the other hand, in *Los veinte y un Libros de los Ingenios y Máquinas*, written for Felipe II between 1564 and 1575, attributed first to the king's engineer Juanelo Turriano and then to Pedro Juan de Lastanosa (Pseudo Juanelo Turriano 1983; García 1989, 33–39; García 1990, 74–137), the book 18 is dedicated to the implementation of the *pilas* of the bridges, which would have cutwaters up and down the stream as *proa* and *popa*. The anonymous treatise will also mention them during the elevation of the work: «que el pilar a

de tener figura de barca con proa y popa, en la proa se rrecibe la corriente del agua y en la popa se despide, y sobre cada una dellas se lebantan dos estribos aplicados a los lados de la puente para la firmeza della» (Anonymous, c. 119, 227). Consequently, piers with their cutwaters on the bed of the river and buttress on both banks are differentiated. But this terminological distinction is not always so clear cut in the prescriptions of the projects. Fray Lorenzo de San Nicolás himself talks about buttress or cutwater to refer to the strengthening of the *cepas* up or down the stream (San Nicolás, c. LXI, 170), and Simón García uses cutwater for both sides of the pier even with different form. Written sources, which use generally both terms, tend to name cutwater to the part against the stream and buttress to the one with the stream. Since Middle Age, the term *cuchillo*[2] is also used, as in the author of *Los veinte y un Libros*. The tip of the upstream cutwater is usually referred to as *nariz* like in fray Lorenzo de San Nicolás, and previously documented in handwritten sources in La Rioja.[3] Less frequent in Castilla seems to be the term *espolón*, taken by Covarrubias in his Dictionary, 1611, as a synonym of *nariz* or tip in the *cepas* and *pilares* in the bridges. Its use is recorded in La Rioja in written form not before the XVIII c.[4]

These notions on terminology, apart from their lexicographical interest, can also be of use when planning the calculation of the thickness of the piers related to their height and the span of the arches, and the necessity, or not, of having the same dimension for those which support the strength of two arches and for the buttress in the banks which support only one. Spanish theorists and builders don't seem to differentiate between the central piers of the bridge and the lateral buttress in the work because their naming is indistinct. This distinction is stated by Mesqui for whom the *Traité des Ponts* by Henri Gautier, 1714, is the first one to differentiate between *piles et culées* in the laws of proportion for the stability of the work (Mesqui 1986, 181). Generally, in historical bridges the same laws are applied for piers and buttress but it has to be taken into account that these usually depend on the thickness the work gets when it emerges from the level of the water because the construction masters strengthen the base with a bigger platform. Let's go over the work of the piers they propose to comment later on the dimensions and forms they design for cutwaters.

On the founding they proceed to the elevation of the pier with the best quality stone, well squared and worked with mattock, settled in plumb line and leveling. The construction takes two systems depending on the preferences of the master of the project: 1 complete solid ashlar in the whole thickness of the pier, in the first courses or up to the spring of the arches; 2. Ashlar work with headers and through stones[5] in the first courses and continuation of the elevation with ashlar to the outside and filled with rubble and gallet or *médula*. The first system is less frequent but it is sometimes a requisite as in the conditions of 1588 for the rebuilding of the bridge in Cuzcurrita upon the Tirón,[6] Figure 1, or in the project of nine piers for two bridges of wooden board upon the Leza and Jubera in 1629.[7] The second system is the common one with variations in the placement of headers and through stones in the first courses. For instance, Juan Raón in the project of 1657 for a bridge in Alberite upon the Iregua, Figure 2, proposes to harness the ashlar work with headers, 5 to 6 feet long —castellan foot—, and 4 to 5 base in each course and in stretches of 6 for 6 in a height of 4 feet, and with two courses of through stones which would cross the whole interior of the pier (Arrúe and Moya, coord., 1998, 1: 499–503, 2: 870).[8] The relevance of the good joining of the work of the piers stated in these examples, can also be seen in the contracts in other regions like in the constructive conditions in Lerma, Burgos, upon the Arlanza, signed around 1573 by very well known architects like Miguel de Nates, Juan de Naveda, Juan del Río Alvarado, Pedro de la Torre o Diego de Sisniega, among others (Cadiñanos

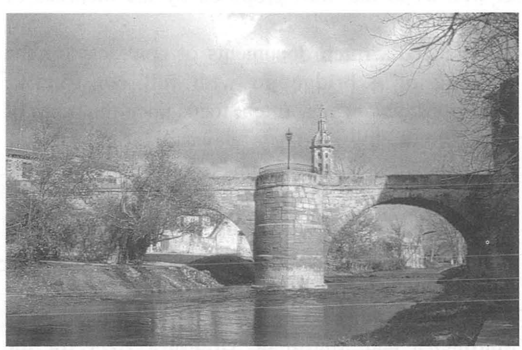

Figure 1
Bridge of Cuzcurrita del Río Tirón, La Rioja

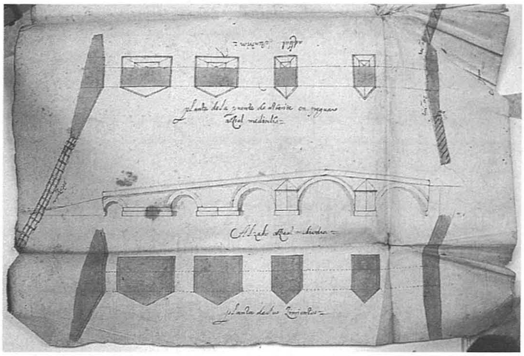

Figure 2
Plan by Juan Raón for the bridge of Alberite upon the Iregua, La Rioja. 1657 (Archivo Municipal de Logroño)

1996, 42), Juan Gómez de Mora in 1619 for a bridge upon brook Abroñigal in Madrid (Corella 1994, 21), and in other ones from the XVIII c. like those in Álava (Azkarate and Palacios 1994, 266–267), or Vélez, Málaga, designed by Domingo Tomás (Camacho 1992, 350). This practice shown in the contracts is advised in the treatises although without specification of small details of construction. In this way, the author of Tratado de Arquitectura XVI c. proposes the use of big stones and a great number of through stones or crossbeams in the base of the piers: «Pero asi la proa como la popa conviene que sean de piedra muy rrecia y que la agua no la gaste y lo de dentro del cuerpo del pilar debe ser hedificado de grandes pieças o muchas atrabiesas porque con muchas firmezas sustiente los conbates del agua» (Anonymous, c. 119, 228–229). In the work by Simón García we find «grandes y fuertes piedras bien travadas y grapadas» (García 1681, fol. 41 v.). Fray Lorenzo de San Nicolás proposes to fill the piers with the biggest stone possible and the heart with good mortar and smaller stones (San Nicolás, c. LXI, 170).

The settling of the ashlar work was made with good lime mortar, usually a mixture of lime and sand fifty fifty which, occasionally, was reinforced with double the lime in the rubble work of the foundation.[9] It is also documented the use of iron staples specially in repairing works like in the ones projected by Juan Ochoa de Arranotegui on the bridge of Santo Domingo de la Calzada upon the Oja, in 1562 (Moya 1980, 2: nº 360), Gaspar de Vega for the bridge in Viveros, c. 1569, with six pounds weight each staple (Corella 1992, 158), and the masters Olate, Pérez de Obieta and Rodrigo de la Cantera in the rebuilding of the bridge in Logroño upon the Ebro, in 1587, where we find staples of one ounce thick, two ounces and a half width and four ounces spigot (Arrúe and Moya, coord., 1998, 2: 853).

The base of the piers is built on a bigger surface than the elevation when it emerges from the water, like in fray Lorenzo de San Nicolás when he remarks that to the piers it should be given good baseboards or *zarpas* «para que queden bien bañadas» (San Nicolás, c. LXI, 170). In the practice of the studied constructions the section of the pier can reach in the base up to four feet wider along the perimeter (1,12 m). In the Treatise by Simón García it is stated, following the rule of Alberti, that the inferior thickness in the pier is double the superior part (García 1681, fol. 41 v.), but it doesn't seem to have reached that proportion in practice. This thickness decreases gradually by means of *dejas*[10] or footing, half foot in each course. This reduction is done usually in plumb line, not being the slope documented in La Rioja until the XIX c. although it was in Álava in the repairs of 1736 of the bridge Marubay in Catadiano (Azkarate and Palacios 1994, 262). As far as the relation between the height and the width of the piers, we can see, with exceptions the preference of extending the pier with its cutwaters once and a half the thickness during the XVI and XVII c., then becoming into twice and a half in the XVIII. Nevertheless, in the project of 1562 for the bridge of Santo Domingo de la Calzada, a length of triple the thickness is proposed although the parameter of the width of the vaults must be consider as well as the measure given to the span of the arch. Related to this, Simón García advices in his work to give for the area of the pier half the surface resulting of the multiplication of the span of the arch for the width of the vault. Of this result, three parts would be given to the upstream cutwater and two to the one downstream (García, fol. 40 v.). In the examples analyzed by Mesqui, French builders tend to the proportion 1/4 for the thickness of the pier in relation to the span of the arch rejecting the extreme 1/6 proposed by Alberti, and avoiding the possible risks in the work (Mesqui 1986, 182). The builders working in La Rioja, as well as the authors of the projects, seem to know the advice of the writers of the treatises, because in the

preserved bridges or in the documented which have been researched, the trend during the Modern Age is to a thickness of 1/3 or 1/4 of the span of the arch. The anonymous treatise on architecture will follow the proposal of Alberti between 1/3 and 1/6, or a thickness of a quarter of the height of the bridge (Anonymous, c. 120, 232), but fray Lorenzo de San Nicolás will take fewer risks when suggesting for the pier a thickness of half the span, proportion we can see in the examples in La Rioja in the XVI c. but in the base or area under the water. The written sources we study do corroborate the implementation of geometrical rules in the constructive practice (Heyman 1995; Huerta 2000), although the knowledge could come from oral sources as the Comissioner of War Marcos de Vierna remarked when supervising some projects for bridges of the XVIII c., and not all the masters would be acquainted to the same extent with the works of the theorists.

The elevation of the piers follows two models used along the three centuries of the Modern Age: 1. lengthening of cutwaters to the grade line forming lay by in the causeway up and down the water; 2. the top in the wall face of the bridge from the beginning of the arches or in different levels in the spandrel areas. The first one is considered as a follow up of the medieval designs but the fact is that the contribution to the safety of the construction, and the space it provided to the causeway and the seat to the passers by, and even the availability for war, commercial, religious or commemorative building, made of it a great success as shown in the constructive practice. This one will be the common model in the north of Castilla and, specifically, in La Rioja, a region of intense activity of building of bridges having the privilege of the passing of the river Ebro and six tributaries to it in spite of the small geography of the area, Figure 3. This model, however, could be ascribed to the practice of masters from Cantabria, around 50% coming from Trasmiera during the XVI c. are documented as active in the region. It went on being used preferably in the following centuries in works done or supervised by specialists from very diverse origin brought in by the contract part or by the Consejo de Castilla. From the XVIII c. on it was kept by local masters, some of them like Francisco Alejo de Aranguren sanctioned by the Academia, which will send other members for the control of the projects to carry out the order by Carlos III in 1777, like Diego Ochoa or Manuel Ángel de Chavarri. In spite of the fact that more erudite models are not unknown, this system is kept in works of certain relevance and new planning up to the end of the XVIII c. like in the bridge of Torremontalbo upon the Najerilla. This was a work carried out between 1790 and 1794 with an intention of modernity, protected by the Real Sociedad Económica de la Rioja Castellana, in the way from Logroño towards the border of La Rioja which would link with the one in Santander and would benefit the exportation of wines, Figure 4.[11]

The second model used in the classic period and proposed by the Italian theorists from Aberti, can take different forms in the top. The most simple is the crown by means of a perpendicular plane to the line of wall faces used by Alonso de Covarrubias, c. 1543,

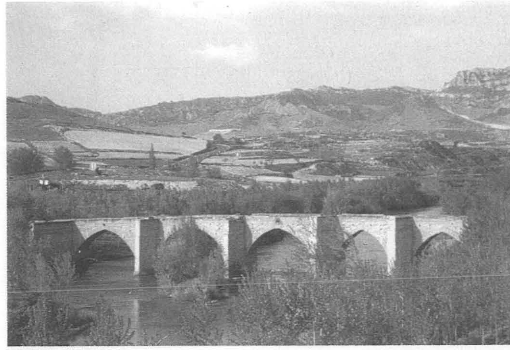

Figure 3
Bridge of Briñas upon the Ebro in Haro, La Rioja

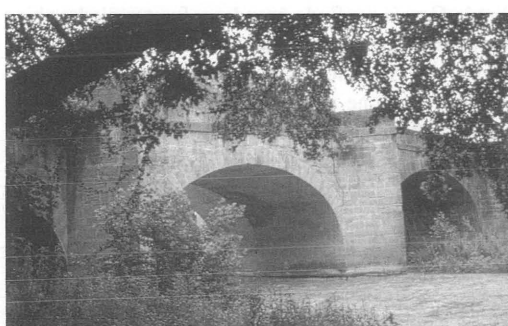

Figure 4
Bridge upon the Najerilla in Torremontalbo, La Rioja

and Gaspar de Vega, c. 1569, in their designs for the bridge of Viveros. But the most widespread from the XVI on will be the one with two planes in double slope following the triangular section of the cutwater as in Juan de Herrera in the Puente Nuevo upon the Guadarrama, Figure 5. The end in conic *chaperon* shape with graded courses which will become general in the designs of the XVIII c., Figure 6, will also be used. It has not been preserved in bridges in La Rioja before the XVIII c., Figure 7, but a triangular cutwater design was presented by, probably, Juan de la Portilla for the Bridge-aqueduct of Zamora in Cervera del Río Alhama, in 1653, later on modified so as to lengthen it to the grade line (Arrúe and Moya 1998, 1: 653–654). In the same way, Juan Raón would use this type of pier in his design of 1657 of the bridge in Alberite, the cutwaters crowned in double

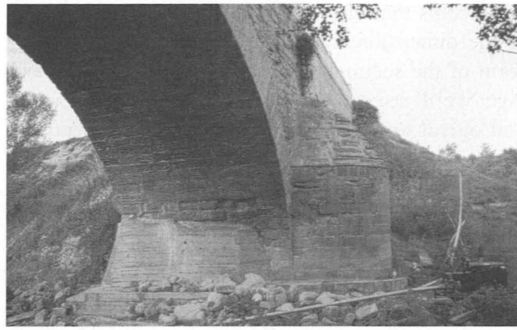

Figure 7
Bridge upon the Tirón in Leiva, La Rioja

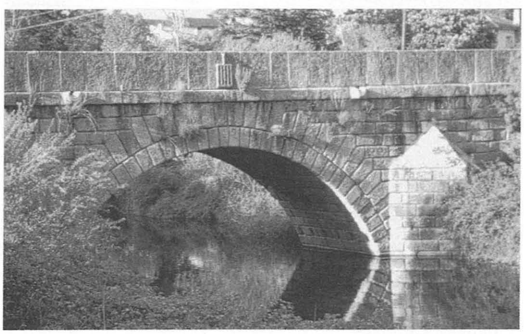

Figure 5
Bridge Nuevo upon the Guadarrama between Galapagar and Torrelodones, Madrid

slope under the parapets to the level of the extrados in the arches in two of them and to the spring in other two. The down stream cutwaters, on the other hand, are crowned with a hood bell shaped and a ball in the top, Figure 2 (Arrúe and Moya 1998, 1: 500–501). The placement of the ball in that area, element related to the architecture in El Escorial, is found in France in the Bridge Charraud upon the Sedelle in Crozant, not finished until 1695, and in the Bridge Wilson upon the Loira in Tours, at the end of the c. XVIII, Figure 8 (Prade 1986, 229, 151). In Spain, from the designs by Herrera on, the balls are placed in the crown of the parapets in plumb line with the cutwaters, as intended for La Rioja in 1639 Domingo de Urruela y Velasco in the bridge of Calahorra, where the balls themselves would be of weight and constructive function, as suggested in fray Lorenzo de San Nicolás (Arrúe 1998, 145).

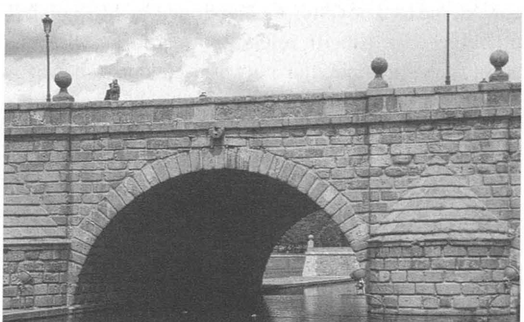

Figure 6
Detail of the bridge of Segovia in Madrid

Figure 8
Bridge Wilson upon the Loira in Tours

It seems that historiography shows more attention to the dimensions or height of the piers than to the form of the section of cutwaters. Along the Modern Age we'll see triangular, semicircular, trapezoidal and ogival up and down cutwaters independently of the continuation or not to the grade line as well as the use of mixed forms combining one and the others in the cutwaters themselves, or different in one or the other. The most frequent and «classic» is the triangular or angled for the tip and the rectangular for the down stream cutwater. The researched bridges with this typology in La Rioja have a variable angle for the tip from the right one (bridges in Ribafrecha, Villanueva y Brieva), the 80° or 82° (Igea, Cornago, Viguera), the 75° (Igea, Cornago) and the 65° (Leza). It seems that the proportion proposed by Alberti, 3/4 of the right angle or, if less salient is preferred, 2/3 of the right angle, is never reached (Alberti, l. IV, c. VII, 188). Fray Lorenzo de San Nicolás prefers the right angle and, however, a not too pointed angle to avoid deterioration as Alberti states (San Nicolás, c. LXI, 170 y 172). The author of *Los veintiún Libros* will prefer the obtuse angle to separate better the waters (Pseudo-Juanelo Turriano 1983, lib. 18: 486–528). The same variability of angles, always inferior to the right one, observes Mesqui in France (Mesqui 1986, 196–198).

Ogival form, also known by Spanish historiography *en huso* and the French *en amande*, is placed by Mesqui in France the area of Limoussin for the upstream cutwaters being the bridges of Saint-Marcial y Saint-Etienne de Limoges, XIII c., the main examples, Figure 9. He relates this form to the military architecture of the same period because it can be observed in towers of ogival section like in the Poiteau, Parthenay or Coudray-Salbart. For him, the hydrodynamic superiority of this typology has not been proved and he understands that this fashion is forgotten and then back in the bridge of Moulins, work by Jules Hardouin-Mansart in 1704 (Mesqui 1986, 197; Lombois 1993, 40–44). Also Prade coincides in signaling the rare use of the ogival cutwater in France, only upstream, during the c. XIII and XIV, and the frequent adoption of the form in the c. XVIII, when it is also used downstream (Prade 1986, 32). If this is the fact in the constructive history of bridges in France, it is not the one in the Spanish practice as we can not locate here contemporary parallels of the medieval French while in the XVI c. bridges of this cutwater form were built. Because of this, its use during the Renaissance more than an answer to tradition can be considered as a novelty. As mentioned by Mesqui, it is probable that Alberti refers to this form when he accepts the semicircular cutwater excepts when it is not so obtuse or blunt to interfere with the speed of the stream (Alberti, l. IV, c. VII, 188). The anonymous treatise on architecture follows closely these rules: «Las popas y propas, si fueren rredondas, saldrán de la obra en figura de medio círculo y si son agudas em punta, saldrán quato la mitad de la anchura de la puente; o según otros, hácense en ángulo que sean dos terçias de rreto» (Anonymous, c. 120, 232). This relation of similarity between the round and the pointed cutwaters, or the ogival form, and the difference with the triangular, angled or acute

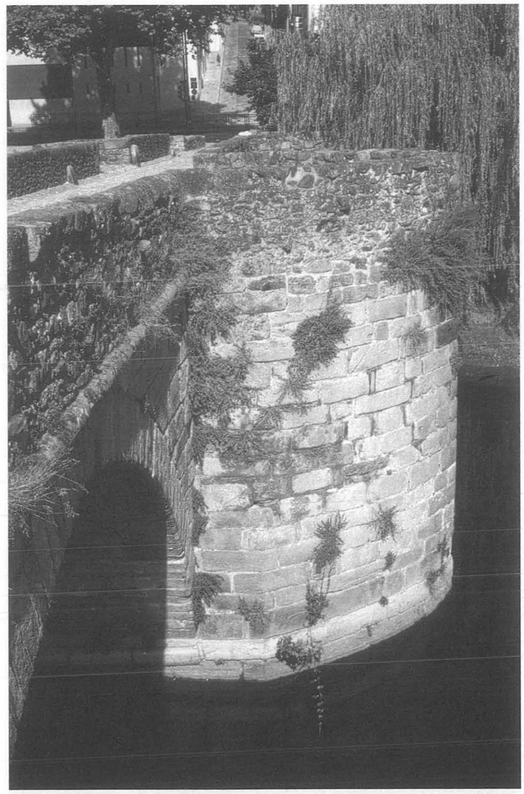

Figure 9
Detail of a cutwater in the bridge Saint Etienne upon the Vienne in Limoges

manifested in treatises, is stated and clarified in the constructive conditions that Juan Pérez de Obieta, Rodrigo de la Cantera, Juan de Olate and Pedro de la Torre Bueras present in 1587 for the rebuilding of the bridge in Logroño upon the river Ebro. According to these masters, it was necessary to substitute some medieval piers with triangular cutwaters for others in ogival form because it was considered that these ones expelled better the water and undermined less: «los taxamares y ángulos dellos se arán como se muestra en la planta que son obrados apuntados para más fortaleça y perpetuidad, porque los que son agudos no son tan fuertes y seguros porque los ofende mucho más el agua quando los yere, ya que el agua no les dane los árboles y maderas que bienen los descalabran por ser delgados y en los rredondos se despiden mexor» (Arrúe 1998, 139–140).[12] A similar reform is undertaken in the bridge of San Vicente de la Sonsierra in 1594, Figure 10, with participation of, also, Diego de Sisniega, and in the bridge of Cuzcurrita, Figure 1. Pedro de la Torre Bueras uses it in his designs for La Rioja (Ezcaray, Torrecilla de Cameros, Viguera), Burgos (Buniel) or Cantabria (Arce), the latter one with Siniega himself and other masters from Cantabria, in 1585. A year before it had been chosen for the Puente Mayor of Palencia by Francisco del Río, Alonso de Tolosa, Juan del Ribero and Francisco de la Puente (González et al. 1991, 633 and 575), with piers of 14 and 15 feet thickness for spans of about 35 feet, which confirm the proportions mentioned above and used in contemporary bridges in La Rioja (Arrúe 1995, 167). But there are examples of ogival cutwater from the end of the XV c., like in Montoro (Córdoba), Figure 11, and its use during the three first quarters of the XVI c. in Castilla and León have been related to the military design by Francesco Di Giorgio Martini (Aramburu 1992, 62–63). It is not surprising then that the ground plane and the elevation of the bridge drawn by Simón García in his treatise contains ogival upstream cutwaters and rectangular downstream ones (García, fol. 41), although its elevation is crowned at the height of the spring of the arches as opposed to the continuation to the grade line preferred by the masters from Cantabria. It is likely that he would introduce in this way the proposals by Herrera, combining both typologies. During the XVII c. the preference for the ogival cutwater is confirmed in some projects in La Rioja —Murillo, 1629; Calahorra, 1637—, and it will be the common one in the projects of the XVIII c. like in those one by Diego

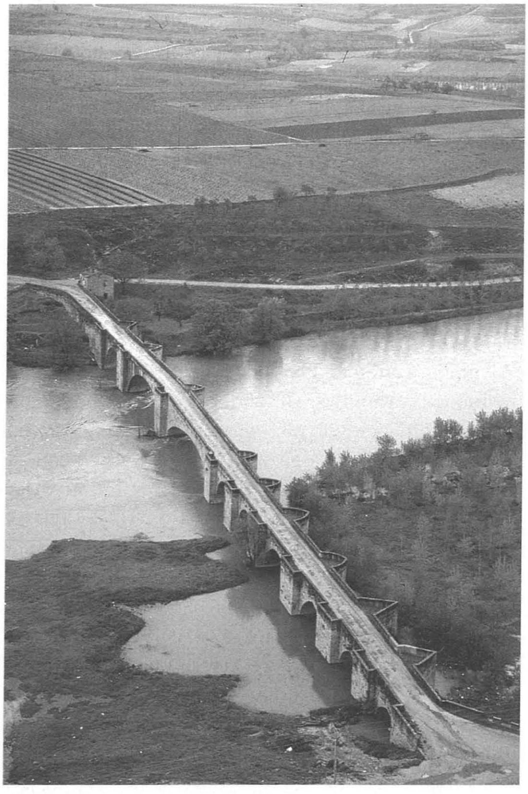

Figure 10
Bridge upon the Ebro in San Vicente de la Sonsierra, La Rioja

Figure 11
Bridge of Montoro upon the Guadalquivir, Córdoba

de la Riva for the bridge in Agoncillo in 1765 and Francisco Alejo de Aranguren for the bridges in Leiva in 1772, Figure 7, Pedroso in 1775 and Miranda de Ebro in 1786. These projects are revised by the Comissioner of War Marcos de Vierna, who designed in 1761 the bridge of Aranjuez upon the Jarama with ogival cutwater up and downstream, crowned with conic chaperon (Andrés 1989, 97–101). They will also be used by, for example, José García Martínez, captain of engineers, in 1770 for the bridge of Vélez-Málaga (Camacho 1992, 348), and we can see them in the bridges of Herreño and Retamar upon the Guadarrama in Madrid (Navascués 1985, 106–107), Figure 12. In the same way, they are designed by Perronet in the bridge of Chateau-Thierry, and with semicircular cutwaters in the ones in Mantes and Orleans (Perronet 1987, 169–171, 121–151, 194–236).

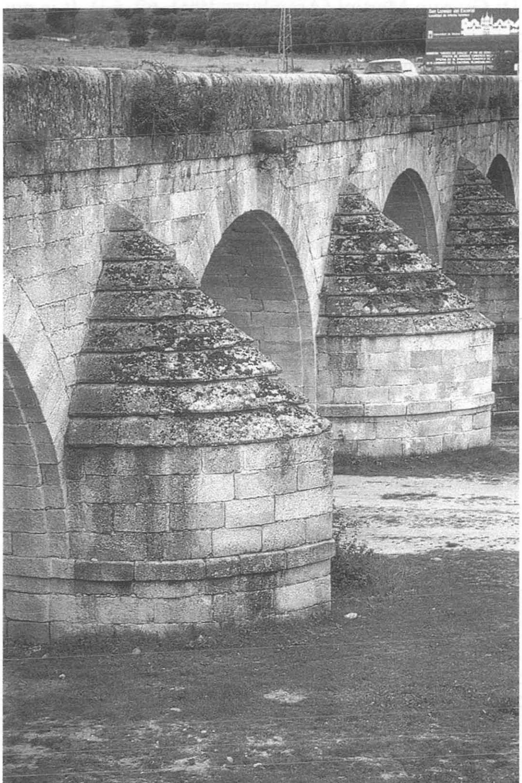

Figure 12
Bridge of Retamar upon the Guadarrama, Madrid

Semicircular shape will be frequent in Spanish bridges under the kingdom of Felipe II, like in Almaraz upon the Tajo in Cáceres, the one designed by Andrés de Vandelvira for Ariza in Jaén or Hernán Ruiz el Joven for Benamejí in Córdoba (González 1998, 124–133).[13] Its use down the stream is found in the mentioned bridge of Montoro, around 1500, and in the projects of the XVII c. by Gómez de Mora and José del Olmo for the bridge of Toledo in Madrid. In this bridge, Pedro de Rivera, in the beginning of the XVIII c., will lengthen the triangular cutwaters with semicircular buttress to the causeway so as to form lay by in it, against the opinion of Teodoro Ardemans (Navascués 1968, 54; Verdú 1993, 59). This combination of forms had already been previously used and one example is the bridge of San Marcos in León, work of the end of the c. XVI by Felipe and Leonardo de la Cajiga, in which cutwaters up and downstream are lengthened with semicircular headers up to the causeway (Fernández, Abad and Chías 1988, 212–217). Less frequent will be the use of the trapezoidal form, planned in 1578 for the Puente Nuevo in París, containing more mannerism than function according to Mesqui (Mesqui 1986, 197). It was used by Lucas Gutiérrez de Bargas for the cutwaters he planned in the Puente de Toledo (Navascués 1968, 55) and can be found in those ones in the bridge of Cornago upon the Río Linares, built at the end of the XVI c. (Arrúe and Moya, coord., 1998, 1: 670–473), or in the unique and contemporary bridge of Vilanova upon the Arnoia in Alariz, Orense (Alvarado, Durán and Nárdiz 1989, 231–237). All the mentioned examples show the variety of opinion hydrodynamics involved, but it must be remembered that repairs on a bridge along history are frequent becoming usual the modification of the elevation of cutwaters, like in the bridges upon the river Ebro en La Rioja. Because of this fact, documentary revision of the implementation of designs and conditions is always open (Arrúe 2000, 28–30).[14]

In the works of the elevation of the piers, the accommodation of the centerings to go round the arches is foreseen by means of corbel or short timbers (González 1998, 122), Figure 13. The contracts of construction do not specify the features of the centerings as it is a work usually sponsored by the council, as well as with the rest of the timbering and the nailing. However, we can find in them the necessity of the good joining of the springers and

Figure 13
Central vault in one of the bridges upon the river Iregua in Viguera (km 313, N-111), La Rioja

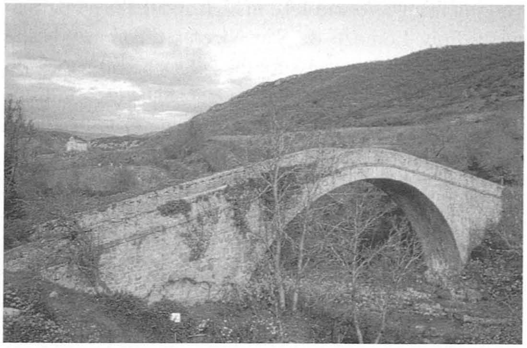

Figure 14
Bridge upon the Iregua in Pradillo, La Rioja. 1771

voussoirs with the ahslar of the cutwaters and buttress, even mentioning sometimes the seat of the voussoir without lime mortar, wedges, grouts or fillers. The action of vaulting of the arches is called *retumbar* or *retumbear*, the spring, *arrancamiento*, *retumba* or *retumbla*, and the arches *tumbas* (Anonymous, c.119, 229). In the Modern Age the most widely spread arch is the round one sometimes segmental, but it was kept along the XVI c. the pointed arch, of rise always inferior to 2/3 of span, and the basket and elliptic became frequent by the end of the XVIII c. For the thickness of the ring called *anillo*, *cabeza* or *batalla*, a measure of beds between foot and a half and four feet and a half is the usual one in the studied contracts (between 0,42 and 1, 26 m). We find differentiation also between the groins, with bigger size, and the voussoir of the rest of the vault.[15] These are usually of 0,56 × 0,84 m, with beds between 0,84 and 0,91 m. In specific examples like in the bridge of Pradillo upon the Iregua, designed in 1765 by Hilario Alonso de Jorganes, director architect of the royal way from Santander to Burgos, it is proposed to give to the bed or trasdos to the voussoirs a minimum of three feet —0,84 m—, four feet and a half for the groins —1,26 m—, split in two, and that in case those large pieces were impossible to find, a new ring was built (Arrúe 1998, 178–181). This bridge finished in 1771 as shown in the inscription, has only one arch of 27,20 m span and it is a clear example of the maintenance of sloped causeways during the Modern Age especially in mountainous areas, Figure 14 (Arrúe and Moya, coord., 1998, 1: 462–470). But in general the trend is the flat causeway and an odd number of arches as indicated in the Italian treatises.

The master builders try to avoid the stress on the rings of the arches with a layer of rubble work, and they project the close of spandrels with another of ashlar of a course or a foot height, and two fingers or two inches of pad projection, as a uniform base in the causeway, marking in this way the grade line sometimes with outstanding rubble work. The author of the treatise on architecture of middle XVI c. devises a similar system: first medium stone with lime and above it thick and big stones (Anonymous, c. 119, 230). Over this layer of stone, the drainage system was settled to the height of the key stone of the arches or in cutwaters up and down stream, rising to the grade line. Marcos de Vierna will asign for these channels a measure of 0,42 m high, 0,56 m wide y 0,42 m pad projection, and the introduction in one impost of 0,42 m high and four fingers pad projection, in the additions Aranguren plans for the project for the bridge of Logroño in 1779. The contracts of construction used to give details of the implementation of the ledges for a good fixing of the work due to the frequent repairs and reforms they went through. Juan Raón will propose the use of iron staples in his project for the bridge of Alberite in 1657, but Diego de la Riva in the one in Agoncillo in 1765 or Aranguren in the one in Torremontalbo in 1778, propose to reinforce the joint by the opening of a hollow, parallel to the vertical of the parapet, later filled with lime and pebbles. The measures of the ledges are projected accordingly with the height of the bridge

varying from 0,70 and 1,12 m high and 0,28 and 0,36 m wide. Fray Lorenzo de San Nicolás recommends the bigger possible thickness for the ledges because they do not only serve to the passers by but to the bridge itself, and points to the form of pedestal, with its base and entablature, and the balls on top of it. For the pavement he advises the paving with hard slab and moderately thick and cobblestone for very used bridges (San Nicolás, c. LXI, p. 171). In the bridges studied in La Rioja, the paving of the causeway, where the slope or convexity would be kept for the best drainage of waters, the characteristic medieval work is maintained: cobblestone pavement. The task is asigned to a professional stone paver, once the spaces are filled with grougs of lime and stone. In general, this pavement was organized in streets or cross streets signaled by lines of stone very big in the center and other smaller ones crossing forming boxes filled with pebbles. This system seems to be common to other regions —for example those ones proposed by Francisco de Arcilla for the bridge in Cacabelos in 1524–1525 (González 1987, 26), or Juan Gómez de Mora in 1619 for the bridge of Ventas (Corella 1994, 21–22)—, although it differs in the measures of the boxes of stone pavement, Figure 15 (Arrúe 1998, 182–187). The width of the causeway tends to vary between the 2,75 m and 5,5 m, dimensions to oblige during the XIX c. to the widening of the boards and the resulting modification of the aspect of the historical bridges.

figFinally, with no time to pose other questions related to the nature of the materials or the decorations of the works,[16] it is worth mentioning the condition expressed in the constructive prescriptions of plastering with lime the wall faces, finishing that has not reached our days. It is documented in La Rioja, for example in the bridge of Nájera in 1694, in the one of The Penitencia in Logroño in 1743, or in the one in Leiva of 1772, and it is confirmed in the conditions of Bartolomé Hurtado of 1668 for the bridge of Ventas, who, among others, proposes to plaster everything with good white lime and good sand (Corella 1994, 27).

All the data above mentioned show clearly the variety of typologies in the elements of the bridge during the Modern Age. Practical experience evolves into a progressive safety in the dimensions of the structure, validated by a theory that even proposes models looking at examples from classic Rome. But, apart from them and from peculiar realizations, those designs which solve the necessities the work presents within geographical contexts in the daily construction, are the ones adopted. That is the reason of the survival of sloped boards or of the lengthening of the piers up to the causeway. Symmetry among the parts and regularity of bonds is the tendency doting with uniformity and weight to the appearance of bridges. Decorations are «modernized» be the bridges from the Renaissance, the Mannerism or the Baroque periods but the inherent dynamic to this type of work does not allow to establish categorical changes in typology during the Modern Age. These changes would materialize and generalize clearly along the XIX c.

Figure 15
Detail of the road in the bridge upon the river Iregua in Villoslada de Cameros, La Rioja

Notes

1. For reference of terminology to handwritten sources, chronology and dictionaries, see the Glossary of terms included in *Catálogo de puentes anteriores a 1800, La Rioja* (Arrúe and Moya, coord., 1998, 2: 936–979).
2. The term is used in the document of obligation of 1377 signed by Mateo Gil for the repair of the bridge of Viveros upon the Jarama en Madrid (Corella 1992, 155). In the repairs of the bridge of Logroño upon the Ebro in 1587 it is planned to take seven courses apart «por la parte del cuchillo» and four «por la parte del estribo». The signature of the archive and the transcription of handwritten sources referred to La Rioja are found in the Catalogue mentioned above. I will refer to it so as to avoid long documentary quotes (Arrúe and Moya, coord., 1998, 2: 855).

3. In the conditions of 1562 for the repairs of the bridge in Santo Domingo de la Calzada, proposed by Ochoa de Arranotegui (Moya 1980, 2: nº 360). García Salinero does not locate it in the *Léxico de Alarifes* until the translation by Miguel de Urrea of the treatise by Vitruvio in 1569 (García 1968, 163). The use of *nariz* is common in the documentary sources in XVII and XVIII c.
4. In the constructive conditions of the bridge of one arch to be built in Tirgo upon Tirón, in 1741, they name cutwater to the ones built upstream and buttress to the ones placed downstream. In the contract of construction of the bridge upon the river Viejo in San Vicente de la Sonsierra, signed by José de Landa in 1751, the term buttress seems to refer to the work of the piers up and down the stream indistinctively (Arrúe and Moya, coord., 1998, 2: 891, 895).
5. In La Rioja, the stone *perpiaño*, the one which crosses completely a wall, is generally called *pasadera* and also the variations *prepiaño*, *pripiaña* and *pripiaño*. The term *pripiaño* is collected by Benito Bails in his Diccionario de Arquitectura, posthumous work in 1802, as *mediana* stone.
6. Signed by Pérez de Obieta, Rivas and Sisniega: «Otrosí serán fabricados los dichos pilares en la manera siguiente todo, ansi en los estremos de la parte de fuera como en las médulas y cuerpos de dentro, de buenas pieças bien esquadradas y galgadas a picón, y con buenas juntas que tengan las pieças de largo a tres y a quatro y cinco pies de largo, y ancho dos pies y dos pies en quadrado, con el alto que tubieren, tenyendo en quenta de guardar sus buenas ligazones, ansí en la parte de dentro como en la de fuera, y echar buenos tizones para que bayan ligando la obra y fábrica de manera que quede con perpetuidad y firmeça, y subirán fabricados de esta manera hasta donde an de començar a nibel los arcos y de allí arriba, subirán por la parte de fuera con sus muy buenas pieças y por la parte de dentro, de froga y buena ripiaçón» (Arrúe and Moya, coord., 1998, 2: 234–243, 859).
7. Following the conditions of the work that the Council of Murillo de Río Leza alloted to Juan de Setién Venero: «toda de piedras lavradas y galgadas para que asiente y aga eleción sobre el enpotrado, asentadas en la dicha primer ylada sovre vuena froa de cal, y su mezcla della a de ser tanta cal como arena» (Arrúe and Moya, coord., 1998, 1: 559–563, 2: 865).
8. The use of through stones in the first course is an expressed requisite in the contract of the construction of the bridge of Prado in San Vicente de la Sonsierra, which José de Landa was obliged to build in 1751: «Y con condizión que la primera ylada de zepas y espolones ha de ser de pasaderas enteras y, en lo restante de todo lo exterior, un sillar entre dos pasaderas» (Arrúe and Moya, coord., 1998, 2: 895).
9. It is condition in the rebuilding of the bridge in Torremontalbo upon the Najerilla by Ignacio Elejalde and Mateo de Retes, contract of 1735 (Arrúe and Moya, coord., 1998, 2: 887).
10. The term *deja*, common in carpentry, is not found in dictionaries of construction until the XIX c. But it is very usual in stoneworks in the previous centuries. With the same meaning of baseboard to reduce the thickness in height we find *retallo* or *restallo, grada, retreta, zapata* and *zócalo*.
11. It was projected by Manuel Echanove with seven eliptic arches between 14,70 y 8,30 m. spand, on piers with triangular upstream cutwater piers and rectangular downstream ones which go up to the causeway (Arrúe and Moya, coord., 1998, 1: 396–404).
12. Confirmation of this rebuilding with ogival cutwaters and the remaining of medieval triangular ones in the bridge in Logroño is shown in the plan Palomares projected in 1849, after the severe damages in the flood in 1775 and before the definitive substitution for the one designed in 1882 by the engineer Fermín Manso de Zúñiga (Arrúe and Moya, coord., 1998, 784–807).
13. There are no rests in La Rioja of examples of piers with upstream cutwaters or semicircular downstream (Arrúe 1995)
14. For example, related to the above mentioned medieval bridges in Limoges, one of the cutwaters in the bridge Saint Etienne presents the inscription 1577, which is not collected by Guibert (1904, 233–234), but it is by Perrier and Bonnaud (1977, 187–188), and it is well known that it went through a very important reconstruction finished in 1619, according to the commemorative inscription. I could not tell if handwritten sources have recently come up with data on the specific works carried out at the end of the XVI c., which clarify the alterations the medieval work could have endured.
15. See the analysis on proportions of arches proposed in the treatises of the time in the work by Santiago Huerta (Huerta 2000, 513–526).
16. It is not preserved in the bridges in La Rioja but in the one in Logroño it was designed a rustic main front with the arms of Felipe II in 1587, and two big stone lions in 1682; pilasters in the one designed in 1735 for Torremontalbo and, in the one in Haro upon the Tirón, they installed two statues with saints on pedestals in 1741 (Arrúe 1998, 188).

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Housing construction analysis in Olivette Park, East St. Louis

Osman Ataman

From 1959 to 1990, East St. Louis, Illinois deteriorated from an «All-American City» to a national symbol of urban blight. Located on the Mississippi River, the East St. Louis of today faces severe economic, social, and environmental problems. Nearly one-quarter of the city's work force is unemployed and about 40 percent of families are living below the poverty level. But East St. Louis was not always a distressed community. With strong ties to St. Louis and the surrounding region, East St. Louis once flourished as the country's second busiest railroad hub. Powerful economic and socio-political forces, as well as unfortunate historical circumstance, propelled the city into a downward spiral that drastically decreased the quality of life in East St. Louis. This paper presents the digital re-construction of the buildings and the analyses of the historical aspects of the housing construction and types in this area. Furthermore, it reports the survey and assessment of the quality of building stocks based on the revitalization plan that will provide some guidelines and suggestions for improvement, stability, and future needs.

Brief history

The settling of East St. Louis dates back to the 1790s. On the Illinois side of the Mississippi, just across from St. Louis, Piggot built a ferry landing and began transporting people across the river in 1797.[1] With the road and river passages, people began settling this section of the country which later became known as the village of Illinoistown. By the mid-1800s, these settlers were producing most of the agricultural products consumed by St. Louis residents. Beginning in the 1830s, coal mined in the Belleville bluffs just east of Illinoistown was transported to St. Louis by ferry. In 1861, the people of Illinoistown voted to change the village's name to East St. Louis to

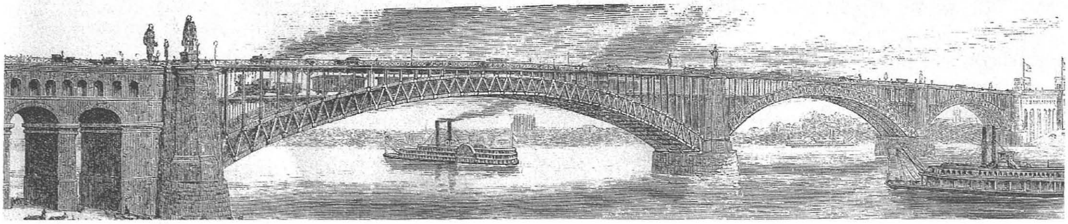


Figure 1
East St. Louis was the western terminus of the East

symbolize the relationship with its Missouri neighbor.[2]

The completion of the Ohio & Mississippi Railroad from Cincinnati to the East St. Louis riverfront in 1857 marked the first of many eastern railroads to establish western terminals in East St. Louis. By the early 20th century, East St. Louis became the western terminus of most of the eastern railroad lines. Advantageous transportation access, abundant available land, cheap coal, and proximity to St. Louis, Chicago, and Indianapolis attracted industries to the East St. Louis area. The years between 1890 and 1930 were known as the «golden era» in the city's industrial history, during which East St. Louis established itself as a major meat-packing, metal-bending, and chemical producing center. These industries promised steady jobs at relatively high wages, attracting an influx of European immigrants and blacks from the South to East St. Louis. The population exploded from about 18.000 residents in 1890 to 75.000 in the 1920s, and it reached its peak in 1945, at 83.000 residents.

Despite its image as a booming industrial center in the early 1900s, certain factors started to develop future economic problems in East St. Louis. Several large plants have closed down or substantially reduced their labor force. Factors behind this trend include the desire to move closer to new market areas; the increasing utilization of truck transport rather than railroads, to reach consumer or producer markets; and the advantages new business operations at new locations rather than modernizing obsolescent old industrial facilities. Nine major industries left East St. Louis between 1950 and 1964, and many middle class families followed their employer's lead.

This economic devastation of the area's industry trickled down to the area's retail and service sector and further eroded the faltering economy and population. In total, the city lost nearly 15.000 jobs and 40.000 people between 1960 and 1990. The number of firms in East St. Louis declined from 1527 in 1967 to 383 in 1987. The city's tax base shrunk from $560 million to $190 million between 1970 and 1990, forcing city officials to cut all but its most essential services.

Olivette Park

Once called «Quality Hill», where the region's wealthiest citizens lived, the neighborhood was severely impacted by the city's economic decline and today faces problems equal or worse in magnitude

Figure 2
Olivette Park in East St. Louis

than the city as a whole. Olivette Park is a 70-block area located near the city's central business district.

Between 1970 and 1990, many of the city's middle— and upper-income families fled Olivette Park and other East St. Louis neighborhoods for the suburbs. The neighborhood's population fell from 5.895 residents to 1.958 residents, while the percentage of families living below the poverty level increased from one-third to one-half. As a result of these changes, the number of occupied housing units in the neighborhood dropped from 1.580 to 595. The decreasing number of commercial and industrial tax payers in the city contributed to the erosion of the tax base and, consequently, the neighborhood's infrastructure, such as streets, sidewalks and parks, deteriorated. Today, many of the streets are missing curbs and sidewalks, more than half of the parcels of land in the neighborhood are vacant, and one-fifth of the structures in the neighborhood are candidates for demolition.

Zoning

Olivette Park is primarily residential, but also contains a mix of commercial, social service, industrial, religious, and public land uses. One-third of the parcels are residential, either single- or multi-family, and five percent are commercial or industrial. This provides residents with a mix of redevelopment options. In addition, there is a surplus of vacant land that could be used for redevelopment. According to the physical condition data, more than half of the parcels in the neighborhood are vacant. The future viability of the neighborhood is dependent upon finding creative and profitable uses for the substantial amount of vacant land.

Quality of building stock

Olivette Park contains an impressive stock of residential and commercial building structures. Sixty percent of the homes in the neighborhood were constructed with solid brick or stone attests to the overall high quality of homes. These structures have largely withstood the test of time, as more than 75 percent of the structures in the neighborhood were rated in good or fair condition in the physical condition survey. The neighborhood contains many large, historic homes, and there has been an increasing interest in historic preservation in Olivette Park. While many of the historic homes in the neighborhood are currently vacant, close to 30 percent are suitable for rehabilitation. Restoring these old homes would preserve the historic character of the neighborhood while providing new housing opportunities.

Land Use	Number of Parcels	Percent
Single Family Residential	402	24%
Multi-Family Residential (1-4 units)	147	9%
Multi-Family Residential (5+ units)	17	1%
Retail/Wholesale	56	3%
Industrial/Warehouse/Utilities/Transport	27	2%
Parks/Schools/Community Garden	60	3%
Social Service/Government/Health Care	32	2%
Church/Religious Facility	20	1%
Mixed Use	31	2%
Vacant	880	53%

Figures 3a and 3b
Current land use

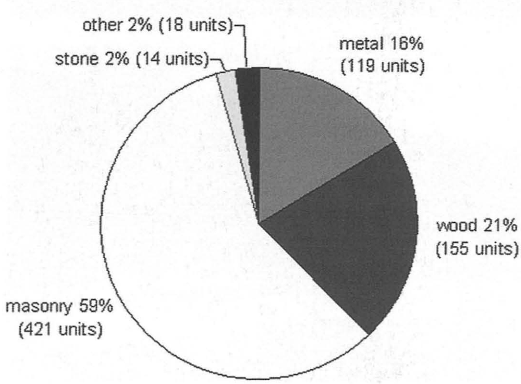


Figure 4
Exterior building materials

HOUSING

Olivette Park experienced a dramatic decrease in the overall number of housing units from 1970 to 1990, as well as the number of occupied housing units. Both the number of housing units and the number of occupied housing units in the neighborhood decreased by nearly 60 percent in those two decades,

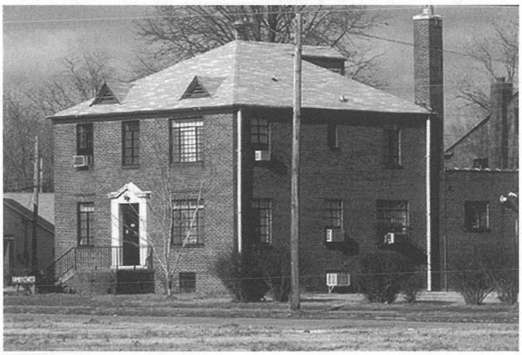


Figure 5a, 5b and 5c
Housing examples in Olivette Park

according to the U.S. Census. Remaining Olivette Park homeowners spend a disproportionate amount of their income on mortgage and housing-related expenses (50% in average). In East St. Louis, on the other hand, homeowners spend about one-third of their income, and homeowners in St. Clair County spend about one-fourth of their income on mortgage and housing-related expenses. Renters in Olivette Park also spend more of their income on rent than those in the city and county, spending 27 percent of their income on rent in Olivette Park in 1990, compared to 17 percent in East St. Louis and 13 percent in St. Clair County.

These high housing costs also often prevent owners from spending money on routine maintenance and improvements, which further erodes the value of the housing stock. A portion of these high housing-related expenses are accounted for by the high property tax rate in East St. Louis. The total property tax rate for FY 1993 was 13.4673 per $100 of assessed valuation. Despite efforts of state and local government to reduce the property tax rate, it remains one of the highest in the state, due to the city's modest tax base.

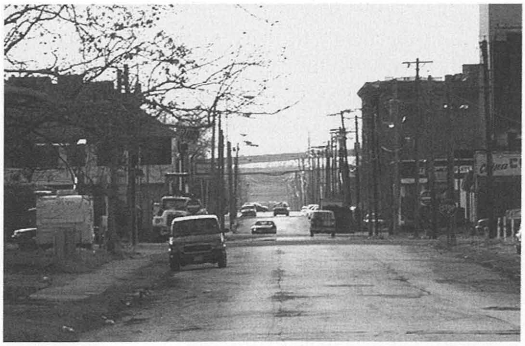
Decaying and dangerous buildings

According to a building condition survey of the neighborhood, at least 83 parcels of land in the neighborhood contain derelict structures. Taking into account that some of these buildings occupy more than one parcel of land, it was determined that 68 structures need to be demolished. At least one-fifth of the structures in the neighborhood are unoccupied. These derelict structures represent one of the most serious concerns of neighborhood residents surveyed. Despite the creation of a new city demolition program, more than 80 percent of residents interviewed rated the city's demolition efforts as poor or totally inadequate.

These kind of derelict structures not only pose a public safety threat on their own, but also threaten adjacent conforming to code and occupied properties. Furthermore, they often function as havens for illegal activity and cause problems for nearby residents.

DIGITAL RE-CONSTRUCTION

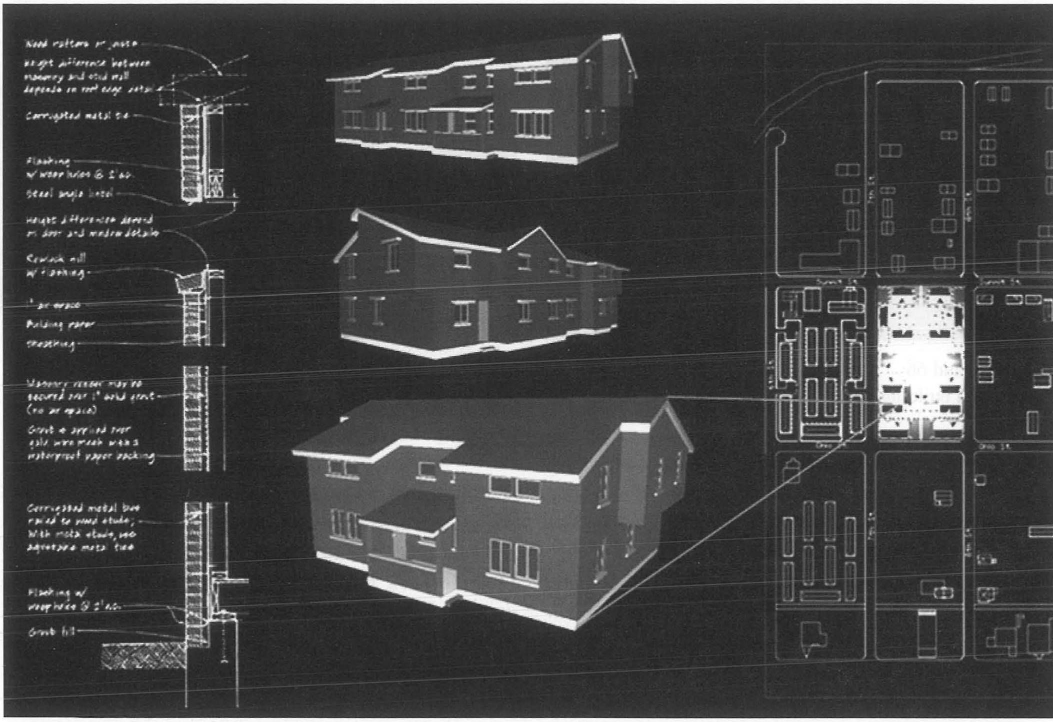
The analysis of an urban area requires a careful examination of the elements, which form that area,

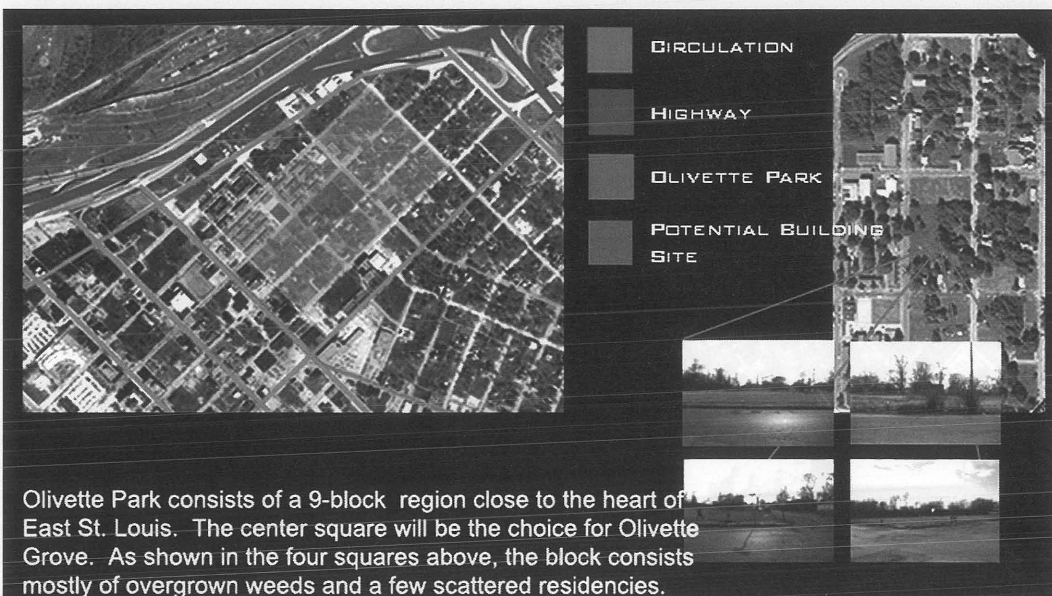


Figures 6a and 6b
Unoccupied building structures in Olivette Park

and the forces that work upon it (Alkhoven 1991). This examination is important because any study in architectural history is best when contextual and can be put in a formal framework only if the interpretations are done from the past to the present. The transformation of an urban area can only be studied in relation to its past for the future. In other words, we need to learn from the past in order to design for the future.

In a like manner, historical analysis study of Olivette Park is best accomplished in relation to its context, East St. Louis city plan. The characterization of the city as a network with individual works of architecture might best be addressed by looking at





Figures 7, 8 and 9
Organization and analyses of buildings for digital re-construction

the individual work in relation to its place in the city's network. Both the architecture/plan and history/context relationships can be examined through mapping. The comparison of these things through time however would be best expressed by using a layered map that gives a simultaneous view of the city buildings' reconstruction through time. History and architecture in their respective contexts could in a layered map be carefully examined as changing elements.

This is an on-going project where the data collection and model-building are still continuing. I believe that the organization and utilization of the non-visual information in an interactive way can make it possible to analyze the historical development of an urban setting. Furthermore, the creation of a digital environment where different interest groups such as facility planners, architectural designers, researchers provide various diverse information may result new directions for theoretical and applied research.

Acknowledgements

A large amount of people have contributed to this project. Among those, my «Spring 2002 Architecture Urban Design Studio» students, Urban Planning Students: Angie Morgan, Patricia Nolan, Eric Stoller and Action Research Group deserve the highest level of appreciation and acknowledgement.

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1. Young 1994, 1
2. Ibid, 1

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Stone masonry in rural sardinian building. Evolution of the traditional building techniques between XIX and XX century

Carlo Atzeni

> Practicality and art are characteristics of popular architecture, which expresses a total life-experience and does not allow for a division between thought and feeling (Norberg-Schulz 1996, 231).

Rural Stone Architecture in Premodern Sardinia: Building and Construction Types.

The territory of Sardinia shows widely differentiated geomorphological characteristics and, especially in hilly and mountain areas, it is marked by the presence of a wide variety of stone materials: volcanic stones such as granites, basalts, tuff and trachytes alternating with metamorphic schists or sedimentary stones like sandstones and limestones.

Rather than the inherent physical and mechanical properties of most of these stones, their systematic use in pre-modern building was the result of the ease with which they could be found near the living areas. This also reduced the work of transportation and improved the land for agrarian use.

In fact stone is the most used natural material in masonry and, from a building point of view, it is the major element that characterises Sardinian rural building, because of the countless varieties which could be found.

In the lowlands, where traditional building was mostly based on raw earth (using almost every where the adobe technique), stone material was only used for the connecting base between the earth and the elevation wall, to avoid dangerous instances of rising damp caused by capillary action. It could sometimes also be used for the crow and surrounds of the openings. In the rest of the island however, all the masonry was in stone, generating a very wide range of buildings depending on the type of stone, the degree of working of that stone, or the laying techniques used. This range is often connected to small parts of the territory.

Despite this multiplicity, the relationship between stone wall construction and other building types belonging to the limited traditional range of Sardinian architecture,[1] seems neither direct nor obvious, but there may be a unifying element in the ways of the communities rooted in it. Moreover the local context establishes a specific culture of material use, depending on the availability of resources.

Anyone crossing the territory of Sardinia will easily find confirmation of the above, and will frequently come across buildings made of stone. These will be seen in the Campidano lowland (even if less frequently) which is based on agrarian economy, where the most popular building type is the traditional big house with its internal courtyard; and also in the mountain areas, where sheep-farming has always predominated and where houses become less broad but taller. Also in the wide hilly belt based on mixed economy, where buildings show some characteristics of the mountain buildings and other characteristics of the buildings of the lowland plains. Examples will be found, too in historic Gallura, in the north-east of

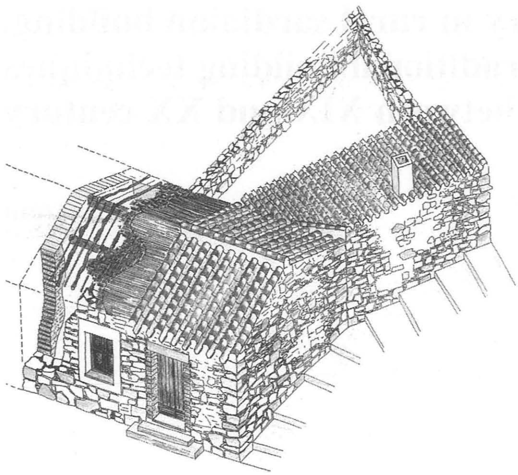

Figure 1
Traditional stone house made of basalt erratic blocks in Bauladu (western Sardinia)

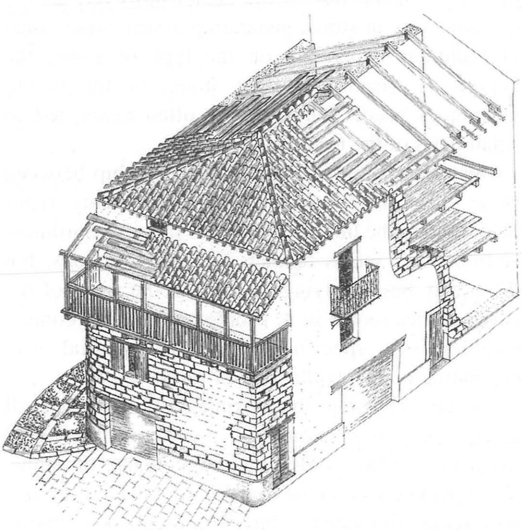

Figure 2
Tall mountain house made of roughly worked granite stones in Gavoi (central Sardinia)

Sardinia, and in the Sulcis area, in the south-west, where the need to defend isolated property has involved the development of isolated types building with particular characteristics.

In the context of rural stone building, although it is not possible to find one constant building type and association, the masonry cell, that is the living area completely surrounded by load-bearing walls (usually maximum 4x4 metres), represents a constant in terms of distribution of structural type of stone architecture.

The elementary cell, which was one of the first advanced shapes of rural building, develops from the archetypal stone building, and adds its regulative unchanging element: the control and organization of space through the principals of juxtaposition and superimposition, and at the some time it is also an efficient solution to structural problems because of its box-like shape.

Parallel and reciprocally right-angled couples of walls carry out two different but equally basic structural roles. In fact one couple has to support the wooden ceiling (usually only one but in some mountain areas there could be even three of them), and also the sloping wooden frame covering, by means of a trilith system; the other couple has to balance the whole system, preventing the overturn of the load-bearing walls by the action of any unexpected horizontal thrust.

From this point of view an important contribution is made by the wooden structures like intermediate floors and roofs that guarantee a certain degree of sharing of the eddy thrust to every wall and also create a stiffening of the whole structural complex, introducing weak restraints that favour its natural settlement and settling well. Moreover, with the specific aim of making the various masonries integral and collaborative as much as possible, limiting their rotations and their independent movement, a widespread routine in order to sustain the floor system was anchoring the beams to the surface of the external walls whit a metal or wooden contrast element assuming that the beams could produce even a static function on axial condition. Either during the construction or later with the same aim, the common system was to fill the inner part of the masonry with metal tie-beams.

Stone building composed of masonry cells is obviously limited in terms of its potential for growth, which comes from the assembling of the elements known as «basic modules» today; moreover as will be shown in detail later, the typical Sardinian wall is not apt to sustain large concentrated weights like big beams. This fact along with the difficulty of finding

wooden elements to cover considerable spans caused a very limited dimensional development of the masonry cells, reducing of the flexibility and the use of the planimetric building system.

On the other hand the structural independence of the various cells favoured the growth and development of the building over periods of time with variations, relating to family needs and to the growth of family settlements. The possibility of sharing a wall creates a considerable improvement in the total structural behaviour without any particular building complications.

Building Techniques and Mechanical Properties of the Traditional Masonry

Traditional Sardinian building is very poor and almost exclusive use of natural materials, such as wood, earth and stone, it is one of its main properties.

The limited economic resources of the majority of rural settlements always forced Sardinian builders to use raw materials found mainly in limited area close to villages. Moreover with the aim of reducing the building costs, investment in the processing of materials was reduced to the minimum necessary for a correct positioning, giving buildings an archaic look.

Building in stone was ruled by the same principles giving them a very «natural» look.

Father Angius, in the first half of the XIXth century travelled the whole regional territory describing every town.[2] When he introduces Villasalto, a mountain town in the historic Gerrei's region in the south-east of Sardinia, underline the above, and with a researcher's exactness typical of his period briefly covers the aspect of building. Using rather untechnical language he said: «All the houses are made of stone, the older shapeless ones look like dens, but the newer ones are really better . . . » (Angius, Casalis 1833, 29).

One of the most striking aspects of the study of typical Sardinian building techniques is the apparent continuity with which the local artisans have handed down the know-how over the centuries (until the first half of the XIXth century and in some places until the Second World War) remaining entirely uninformed about the modern technological revolution. The isolation of the island settlements was certainly the greatest obstacle to the circulation and spread of techniques coming from the mainland. However a thorough study of Sardinian historic events, indicates that Sardinian building shows many similarities and convergences with other Mediterranean areas despite its strong local character, and it has rooted its origins in the distant past.

From this point of view the case of stone masonries is symbolic. In fact the typical shape used is made by the simultaneous construction of two parallel stone faces, between then there is a hollow space filled with earth; little stones and brick crocks.

Because of the weakness of the filling stability of the wall depends on the passing elements (diaton or header) which connect the two faces giving them a certain level of stability. This function is obtained by the notable dimensions and the head position of these elements.

The oldest masonries were dry-built as were, sometimes, even those ones built during the all XIXth century and, in certain cases, also the ones of the beginning of the XXth century. For this kind of masonries the only flattening system between the raw or rough hewn blocks was entrusted to the use of earth. To create a more efficient contact between the uneven surfaces of the stone elements, little stones or brick chips were placed into interstices between the ashlars, limiting the washing away of the earth inside the central core.

The stone masonry building types in Sardinia are certainly not an innovation, they are easily traceable to the *émplekton*, which Vitruvio mentioned in the second book of the De Architectura[3] about two thousand years ago.

In that book the most important aspects, which are also features of Sardinian masonries, are analysed by the roman architect who describes the stone filling technique as a building routine widespread among the rural buildings of Latium, while he assigns to the Greeks the use of passing elements (called *diatonoi*) crossing the full width of the masonry.

In the first years of the XXth century, Giovannoni describing the roman masonries identifies among the concrete works, a particular small masonry, very close to the Sardinian ones, and about its filling he says «it might be said to be made of unshaped stones or pieces of brick, which means made by elements not cast placed but in irregular hand worked layers . . . » (Giovannoni 1999, 22).

Figure 3
Section of a basaltic ashlar masonry in Paulilatino (western Sardinia)

The dry laying without the help of binder or mortars, a typical characteristic of stone masonry in premodern Sardinia, would surprise nobody if we consider the skill that the Sardinian builders have shown to possess in this type of building since remote times. The hundreds of *nuraghi* spread in the regional territory and still perfectly preserved, are good witnesses of this fact.

The differences in the masonry of different Sardinian historic regions are caused, less than by the basic building techniques, then by the wide variety of the stones used or by the less refined level of work, which introduces considerable variations in dimension, weavings and static capability of the masonry.

In Barbagia, a region near the Gennargentu, the principle Sardinian mountain range, grey granite is the most widespread material, but not the only one, used in masonry, relating to two schemes easily assimilable to the uncertain work or to the *pseudoisodomon* work.

As it is well known, the first one, older and more elementary than the second, comes from the use of erratic blocks which vary in size and shape, without any sharp edges and which are little worked, during the building attention was seldom paid to the toothing between faces and between the filling core and diatons.

This masonry type shows frequent cases of the mixture of different materials that, sometimes, assume very singular shapes and behaviour.

In fact, besides the use of different kinds of stone, we find some cases, such as in the town called Sarule,[4] where the wooden elements, crossing the whole width of the wall alternate with stone blocks, instead of the stone diatons, or they are placed along the length of the face even for some metres having the same function as the stone blocks.

Relating to the formal and dimensional homogeneity of the elements it is possible to see further sharing between weaving of loose stones, with sub-horizontal courses, or weaving made by rough hewn ashlars taking turns with erratic blocks, etc.

The second scheme is referable to the pseudoisodomic work made by rough hewn blocks, the recurring dimensions of which, 50 × 18 × 18 cm, still have the proportion of 1:1:3 analogous to those of the blocks used by the Romans to make masonries with work-stones.[5]

The ashlars length defines the maximum thickness of the wall and therefore the two parallel faces are spaced out.

The joint is very efficient and in each layer it is guaranteed by a passing element placed every two or three blocks lengthways.

In both cases the constituent elements were flattened with earth's mortars and lime, and interstices among the blocks were filled with stone or bricks chips.

A wall made of blocks is more coherent, stronger and firmer than one made of erratic block, it is less thick (about 50 cm), and it also has much better static behaviour especially near some critical points where there are concentrations of strain such as in angles, in junctions between right-angled walls, in openings, etc.

In contrast, in the Gerrei and Ogliastra regions, in the south-east of Sardinia, mansories were built mostly using sedimentary metamorphic schists, the behaviour of this kind of stone, which has a natural inclination for breaking along regular and reciprocally parallel fracture planes, produces erratic blocks with a laminar shape that are very useful for the building of faces made of sub-horizontal layers. Schist-elements were alternated with big blocks, usually granites, and here again the interstices were filled with little chips flattening the placing plane and avoiding the washing away of the earth filling.

Barigadu and Montiferru show different lithologic particularities, because the first area has red volcanic stone called trachytes (the case of Busachi is a fine example), and the second has black basalt; however in both areas techniques and masonry weavings look very similar and they are characterised by large sized, rough hewn and cramming elements, even though basaltic erratic blocks have a solid shape and they are also hard to work, whereas trachytes have an anisotropic crystal structure and they have more lengthened elements, it is easier to rough-hew them and for these reasons they are more useful in masonry.

In these cases the main thing that better characterises urban centres is the substantial monochromatic style related on the stones used: villages in trachyte areas having red colours and most villages in basalt areas having dark stern hues.

A very interesting example of Sardinian stone building is the town called Serrenti. Although it is placed on the edge of the Campidano plain (where raw earth is the most frequently used construction material), it boasts several quarries of pyroclastic rocks, called (in Sardinia) Serrenti stone, particularly suitable for masonry building.

Since the middle ages Serrenti stone has been used to build defence and places of worship, and by the XIXth century also for private houses. The reason for this widespread uses is its properties such as great resistance to strain, and especially its easiness to extract and to work.

In Serrenti to the masonry has two faces, and the different level of stone working presents four differences in the weaving: in chronological order from a period before the XIXth century to the whole first half of the XXth century, we find masonries made of unworked stones, then by rough hewn stones with little chips, then by rough hewn blocks and finally by blocks with face of work-stones flattened by lime mortar.[6] In Serrenti wall thickness is in general about 50 cm, even in the oldest pattern, thanks to the excellent shape and resistance of this kind of stone. This is also to be seen in other areas where the working of ashlars is easy. In more recent masonries with work-stone faces the anomalous proportions of the blocks, having recurrent dimensions of 25 × 25 × 30 cm.

The wide variety and extent of the art of stone building, art in the region is not limited to these significant examples; in fact it is worth mentioning masonries made by pink granites in Gallura (north-east) where the building technique is very close to that used in Barbagia, using volcanic tuff in Meilogu and Monteacuto (in the middle north), limestones in the Cagliari area, sandstone and sedimentary metamorphic schists in the Marmilla and Parteolla area (hilly western regions near wide plains). All of these examples increment the very rich range already indicated.

As already seen, the stability of the masonry cell is based on solidity and the collaboration of load-bearing walls and wind-braces walls.

In a building system made of discrete elements (such the one where stone ashlars are used), difficulties which could be found when attempting to guarantee perfect toothing between mutually right angled walls make the junction, and especially in corner areas, one of the most crucial structural points for the whole stone building.

Sardinian builders knew that problem very well, and despite economic limitations, they were able to find simple but quite efficient solutions.

In the whole region, irrespective of the kind of stone used, the recurring technique consisted of conforming the junction using large sized and well squared blocks being careful to overlap them alternately lengthways or header, so obtaining a close fit of the stones and also good joint-stagger.

In works with rough hewn ashlars that system was the logical outcome in masonries, but for masonries of uncertain quality it was a necessary strong point.

In corner buildings only an alternative solution was possible: that of not doing the junction but linking the

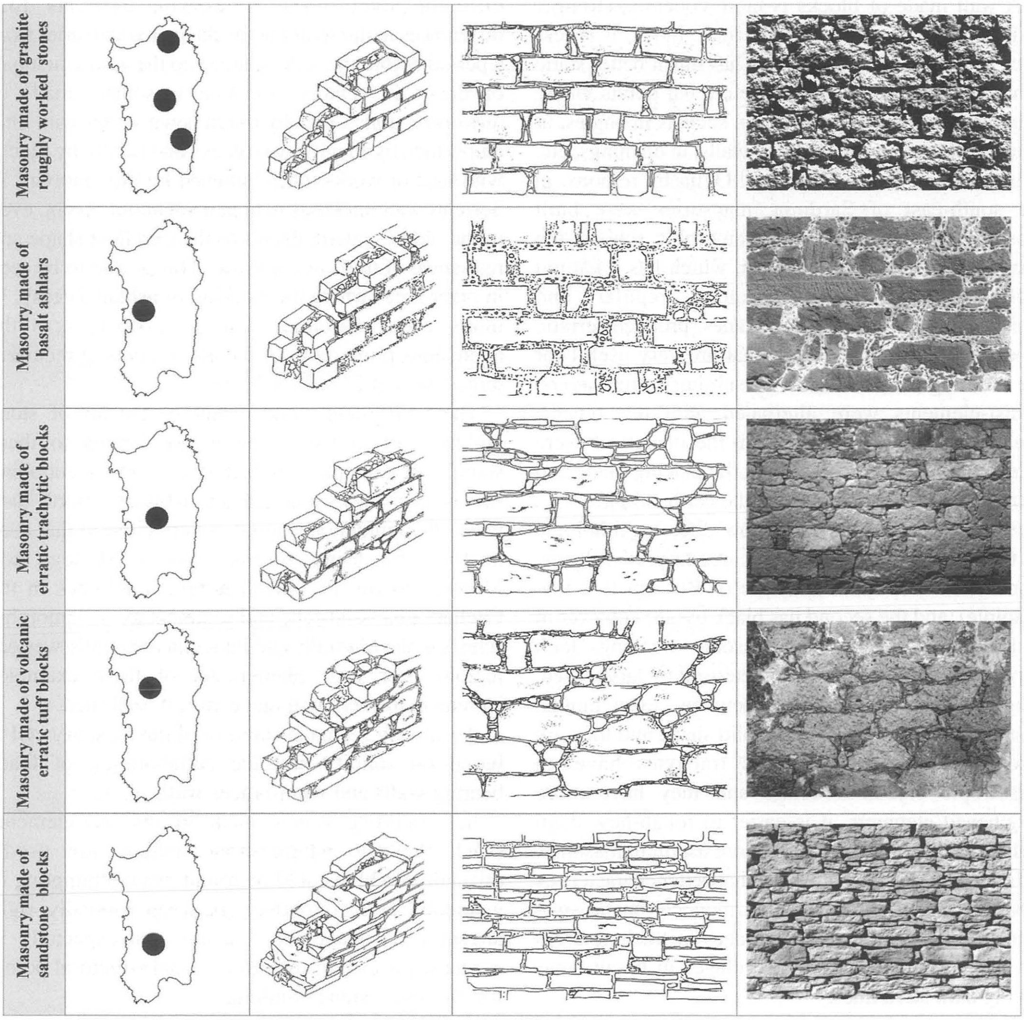

Figure 4
Traditional Sardinian masonry synoptic table.

two right-angled walls using a circular portion with a narrow bending radius. However the use of this technique did not become widespread because it is not efficient. In fact the circular portion does not guarantee the contrasting action between the two walls opposing causing an obstacle to the relative transmission of thrust, and it also generates problems for management of the inner space which is already limited.

As it is well known that two basic aspects for stability and solidity in a masonry are monilithicity and coherence.

Both of these characteristics belong to masonry made of modular elements, with an high level of jointing, with opportunely staggered joints placed on regular planes. The principles of modularity and correct placement guarantee that a wall's load-bearing ability is directly related to the inherent mechanical properties of the materials used.

Figure 5
Corner solution with basaltic ashlars in Paulilatino (western Sardinia)

Examples of this are masonries made of adobes and squared and well linked stone ashlars which are for the static more efficient with parity of load, than walls made of two casually joined faces. Because of the building method, this second type of masonry mentioned above is not very coherent or monolithic which implies that resistance and stability depend the shape and to the bulk of the whole masonry, rather than the mechanical properties of the stone.

In fact, even in case of buildings with only two floors, these structures always show considerable thickness (seldom less than 70 cm and they can easily be 1 m or more).

Moreover the shape created by three heterogeneous vertical layers is not the best one to sustain the concentrated loads coming from the wooden beams. Any localised downward pressure causes damage to the wall in which the two sides to break away from the loose central part. This has a dangerous effect on the stability of the wall. In fact to avoid these mishaps the most able builders used systems of load sharing consisting in wooden joists or the use of stone saddles. In the first case a single wooden element placed at the top of the wall takes the weight of the beams at a right angle to it, whereas more simply in the second case each beam was placed on a big squared stone ashlar that guaranteed the transmission of strain to the surface of a larger resistant wall, thanks to its solidity and size.

A wall's massivety along with limited coherence makes for a lot of difficulties in making openings which are usually narrow and small. In any case the system of openings is one of the most particular elements in Sardinian architecture. The continuity of masonry over the opening was restored by wooden or monolithic architraves in the oldest buildings often using unloading systems such as triangles made of two ashlars placed in contrast, or brick arches; while recently, arched structures (made of stone or brick), have been widely used. Usually jambs were made of square blocks well fixed into the masonry; in some areas they were made of only one monolith placed vertically or more frequently by a system of three large sized ashlars, where the third one was interposed horizontally with the aim of optimising the anchorage to the wall.

The growing use spreading of brick blocks to build all openings, often achieving highly expressive and decorative results, from the end of the XIX[th] century should also be mentioned.

Tradition and Innovation in Masonry at the End of the XIX[th] Century

The XIX[th] century is a decisive period for the development of building techniques in Sardinia. In fact the need for the Sardinian Piedmontese reign to

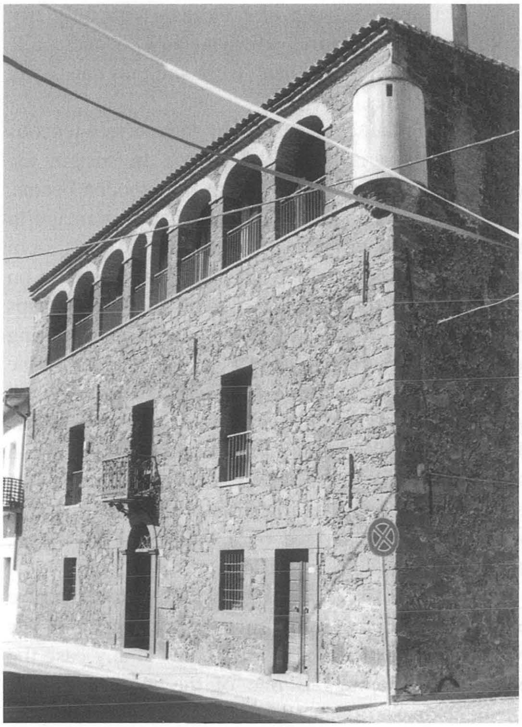

Figure 6
«Tax collector's house» in Paulilatino (second half of the XIX[th] century), currently historical museum

manage the whole regional territory led to the construction of several institutional and military garrisons, such as town halls, barracks, district prisons; all built under the strict control of the Military Engineer. To this end engineers and architects, belonging to the technical corps, arranged several restoration projects, enlargement and readaptation of existing manors, they used a scientific strictness that came from the handbooks which were characteristic of the polytechnic schools established in Europe and in the north of Italy.

These engineers, firstly designers and then people in charge of the works, were professionals who were quite new to Sardinian building which, up to this period, had relied on the substantial concurrence of owners, planners and builders (even if a certain level of specialisation was already widespread among the building workers). Each work was scrupulously documented through written design (design, technical-descriptive reports, quotations for work with sections such as works, quantities and costs), and also through a series of prescripts and advice about the carrying out of works and the about entrepreneur's duties towards the royal administration.

The project documentation, which is hard to obtain,[7] describes some building methods previously unknown in Sardinia indicating innovation. Also for the first time they encode with accuracy working methods and technical devices which together characterise the oldest Sardinian material culture, providing us today with valuable evidence about technological mingling that influenced building practices in that period.

In this precise historical moment, which marks the passage from the premodern and the modern building ways, significant events of architectural and building influences can be seen and a very close link between innovation and tradition emerges.

However, the extremes of this apparent dichotomy are far less than it would seem, and rather than in terms of opposition, these extremes may be explained as an event of the evolutionary process of building techniques, even if a very important one, during which, innovation never overwhelms tradition, and they succeed each other with substantial continuity and technical-material compatibility.

In the same period a larger circulation of specialised labour from the peninsula may be noted, and this contributed in a decisive way to the development of new kinds of stone masonries.

An example of this, ones again, is to be seen in Serrenti, in fact around 1880 a little community of Tuscan stonecutters coming from Montelupo settled there and really influenced local artisans.

Some of the chief new elements for masonries are clearly described in September 1845 by the architect G. Pau, designer and manager of the enlargement work for the «Regie Carceri» in Osilo, in northern Sardinia. In fact in art. 3 of «Istruzioni e capitoli d'appalto» the material and structural quality that masonries must have, is specified. In particular: «We would archive the building of new masonries that are going to be made of hewn stone, and ashlars with double ways in the external corners, and using a good cement made of melted lime and white albino, in the proportion of 1:3 for the last one and 1 for the lime». And in art.6: «Every curling and piering of the

masonry will be done with a good mortar of melted lime and white albino following the right rules of the art».

In this part there are the directions for finishing the masonry and its protection with a protective coating of plaster.

In this period similar executive precepts complete most of the projects for public works.

As a result the using of the rough hewn stone and of the ashlars spread quite fast, which caused a general optimisation of the mechanical characteristics of the masonry, due to more suitable use of the stone materials.

This spreading was however, limited to public buildings or to residences belonging to quite well-to-do families, except in towns in Gallura and in Barbagia, where the stone working concerned also rural buildings.

In the rest of the island masonries were still made by roughly worked materials, but it became systematic to use well squared stone to build particular points such as arrises and fixed, joints between wall, jambs of doors and windows, crowing belts, etc.

Moreover the use of binder made of lime, instead of the traditional one made of earth, introduced a notable improvement in the coherence and solidity of walls and also a greater resistance to washing away caused by rain water.

Figure 7
Masonry made with worked trachytic stones in Busachi (second half of the XIXth century)

Figure 8
Pseudoisodomic work made with trachytic ashlars in Busachi (first half of the XXth century)

It can also be seen in the choice of walls of working materials and of binding with relation to the function that masonries had in the structure of buildings. For this reason, boundary walls were made of masonry with erratic block which was prepared dry and flattened by earth, while perimetric and internal load-bearing masonries were made of ashlars or by walls of well worked blocks joined by cement made of melted lime, with quarry or beach sand and with white albino.

Also plastering of the masonry faces with lime cement, whether inside or outside, with the aim of protecting the wall, became a building routine in that period, even if it should be remembered that plastering based on earth was previously widespread.

The assessments of that period underline the high cost of masonries compared with other items. For example, in1848 for the enlargement of a private residence in the centre of Dorgali (in the east of the island), with the aim of turning to barrack called «Caserma dei Cavalleggeri», the whole work foresaw the building of large areas of roof and also a good deal of finishing work, but the influence of masonry

was valued in cost higher than 30% than that of the other items by the designer architect Galfré.

In fact, out of a total charge of 2900 «Lire Nuove», masonry building with plastering and painting amounted to slightly more than 1000 L.N. with a unit price for just the masonry building of 9 L.N. for each cubic metre, 0,30 L.N. was added to this price for each square metre, for the chocks and splintering, and finally 0,45 L.N. for each square metre for plastering and colouring with milk of lime.

The whole building process was always inspected by the works chief, who, with reference to the precepts written on the contract, had the power to demolish or to have rebuilt any works did not meet the requirements of the design.

The compulsory character of the contractual articles did not exclude masonries, which continued to be the crucial element of the building, and for this reason they were object of particular care.

In fact, several times articles of historical specifications, concerned with masonry, mentioned the high quality required for stone materials also the correct laying techniques related to the best rules of the art.

Moreover as specified in the art. 10 in the contract tender for the «Caserma dei Cavalleggeri» of Dorgali, the role of control of masonry quality by the works chief was underlined. Here, finally, it said «no plastering on new masonries was allowed before the check of the work and of the materials used . . . ».

It should be mentioned that at first only public and military buildings, were involved in the sudden changes and only later and gradually did these involve the rest of residential buildings.

Moreover, often big traditional residences were attributed institutional functions. These were adapted to the new demands with limited improvements leaving the original typological and constructive characteristics untouched.

There was a change but one feels it never assumed a revolutionary nature, in fact the building materials did not change, merely improved the working methods some techniques of laying, without forgetting the consolidated experience of local know-how.

It is really the integration between old and new which is the most important character of this period. This contributed to enriching the poorer and archaic elements of traditional Sardinian building, also from the point of view of decoration.

Never has attention been paid so much as in those times to detail, and for masonries this generated more care in doing relief band course, end cornices with particular shaped ashlars, refined corner solutions, intrados with affected and perky openings, often made using stone and brick materials in turn.

Through the matching of traditional building techniques and imported ones, the list of stone masonries in rural Sardinian building has moved towards solutions typical of urban architecture, giving the buildings of smaller towns a marked urban feel.

Notes

1. For futher information on building types in historical-traditional Sardinian architecture see: Le Lannou 1941, Baldacci 1952, Angioni & Sanna 1988, Mura & Sanna 1998.
2. Around 1830 Father Vittorio Angius waited to write the chapter about Sardinia, for the «Dizionario Geografico Storico Statistico Commerciale degli Stati di Sua Maestà il Re di Sardegna», he received this work from master Goffredo Casalis who was the administrator for the whole work edited in Turin in 1833 by the bookseller Maspero.
3. Vitruvio, 1997, 148 second book.
4. See Forma 1993–94.
5. «Masonry made by ashlars, *quadrati lapides*, in not really frequent cases when they are wall frames, follow systematically Etruscan technique of the isodomic placement, of the shape of parallelepipedal blocks in which width is equal to height and length is double or triple . . . » Giovannoni 1999, 17.
6. For futher information about Serrenti's masonry see Bellu, Mereu, Pani, Serra 1998–99.
7. At the Archivio di Stato of Cagliari —fondo Tipi e Profili— are stored historical papers of the following public works: «Ristrutturazione a ampliamento delle Carceri Mandamentali di Sorso (September 1845), di Osilo (September 1845), di Ittiri (March 1845), di Ossi (October 1845)», «Trasformazione del Palazzo Baronale di Buschi in Caserma dei Cavalleggeri (July 1846)», «Adattamento della Casa di Mauro Loi in Caserma dei Cavalleggeri (March 1848)».

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Do we need to teach Construction History? Experiences of the School of Housing, Building and Planning, Universiti Sains Malaysia, Penang

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Expertise in construction history has been considered as an important requirement in the conservation of heritage buildings and monuments in several countries including Malaysia. In a building conservation process, the understanding of historical background, diagnosing of building defects as well as analysing of building structures and materials are the critical components to be considered by the project team. Inputs from construction history would facilitate this process by providing documentations of the building history, structure and construction methods. Arguably, most conservation projects would engage conservation experts and skilled workers to ensure quality workmanship and minimum disruption to the original building structure and fabric. However, technical expertise in building conservation is still lacking in Malaysia. Despite this shortcoming, the government has taken great efforts to protect and conserve heritage buildings and monuments in the country primarily to boost the local tourism industry. At any rate, fresh graduates with knowledge and interest in construction history have been in great demand to work in conservation projects. Such circumstances have highlighted the growing importance of construction history as a field of study in the built environment.

The School of Housing, Building and Planning at Universiti Sains Malaysia, Penang is one of the schools that offers undergraduate and postgraduate degrees in the built environment including architecture, building technology, urban and regional planning, interior design, construction management and quantity surveying. Since its establishment in 1972, the School has offered as many as 10 courses related to building construction. However, the subject of construction history has never been taught as a course by itself. Instead, it has been incorporated as part of the syllabus in other related courses such as building construction, structural engineering and measured drawing. In tune with the growing importance of building conservation in the country, this paper will assess the need to establish courses in construction history at the School of Housing, Building and Planning, Universiti Sains Malaysia. A survey and interviews with students and academic staff will throw some lights on this debate. The paper also highlights several challenges of construction history in teaching and learning in built environment. Students' perceptions and expectations of the importance of construction history education are also discussed in the paper. Findings from the survey and interview would serve as an important input to determine the demands for construction history courses as well as to ascertain the future of construction history in Malaysia.

Introduction

Malaysia has a great number of heritage buildings, which are still intact. Most of these buildings were built when Malaysia was being colonized by the

Portuguese, Dutch and the British between 1511 and 1957. Some are still in use as they were and some have been converted for new usage. In order to protect and preserve these heritage buildings, effort has been taken by the government by listing some of the buildings under the Antiquities Act 1976.

Ahmad, A.G. (2002) found that even though the government has taken an effort to preserve the buildings, there are still lacking of skilled labourers and technical experts in the field, particularly in preservation methods and techniques. Therefore knowledge of historical background in construction is important.

This paper focuses on the experiences in teaching and learning of construction history in the built environment at the School of Housing, Building and Planning [HBP], Universiti Sains Malaysia [USM], Penang, Malaysia. The paper examines several key challenges faced by the School such as restructuring the built environment courses in tune with the construction history requirements. Findings from a students' survey also highlight the level of the importance of construction history education at the School.

Overview of Built Environment Courses in Malaysian Universities

Built environment can be understood as an inter-disciplinary field of study that is concerned with the construction, maintenance and preservation of structure and infrastructure that man built. This discipline is usually comprised of architecture, construction, engineering, facilities management, interior design, landscape and urban planning. Other fields of study including environmental science, botany, geology, conservation studies and history are also pertinent in providing critical inputs towards the understanding of the built environment.

Several Malaysian universities have offered built environment courses over the last decades. The University of Malaya [established in 1969] offers degrees in architecture, engineering, building surveying, quantity surveying, and estate management. The University of Technology Malaysia [established in 1975] has programs in architecture, engineering, quantity surveying, landscape architecture, and urban and regional planning. While the International Islamic University [established in 1983] has degrees in architecture, landscape architecture, and urban and regional planning. In future, the number of local universities offering such courses is anticipated to grow as the demand for higher education continues to escalate.

The School of Housing, Building and Planning [HBP], USM

Established in 1972, the School of Housing, Building and Planning at Universiti Sains Malaysia, Penang is one of the many schools in Malaysia which is focused on studying the built environment. At present, the School offers six undergraduate programs which are architecture, interior design, building technology, quantity surveying, urban and regional planning, and construction management. Post graduate level by research and taught are also offered at the School including landscape architecture, urban and regional planning, building technology, housing and project management. The School also offers the Bachelor of Architecture degree, which is a five-year architectural program. Subject to demands, other related degrees may also be offered in the near future including tourism planning and estate management.

Currently, there are 879 students enrolled at the School [728 undergraduates and 150 postgraduates]. The School has about 70 academic staff including 3 Professors and 14 Associate Professors. 60% of the academic staff are PhD holders. There are 30 support staff employed in administrative, laboratories and workshops. Since its establishment, the School has been actively involved in many researches and consultancies in the built environment including coastal zone management, building construction, low-cost housing, squatters re-settlement, sewerage, transportation, heritage conservation, tourism, landscape and spatial/Geographic Information System (GIS) applications.

Distinct from other built environment schools in Malaysia, the School of HBP offers a curriculum that is more broadly-based cutting across both professional and interdisciplinary bodies. In doing so, students of HBP are able to draw upon many different disciplines relevant to the field of housing, building and planning during the course of their studies. Students of urban and regional planning, for instance, are required to take basic courses in environmental science, structure,

infrastructure and project costing apart from the core urban planning subjects. Similarly, students of interior design are required to sit for introductory courses in applied statistics and project management before they could embark on their specialized design-based subjects. In essence, both the structure and content of the School's curriculum reflect the major objectives of developing integrative and creative knowledge and skills across a broad spectrum of fields dealing with the built environment.

Table 1 shows the interdisciplinary courses worth 34 credit hours which should be taken by all students during their first year at the School of HBP. Students are only allowed to specialize in their designated fields of study during the second year. Table 2 depicts the full structure of the degree programs offered at the School at the present time.

Teaching and Learning Construction History at the School of HBP

Since its establishment in 1972, the School has offered more that 16 courses related to building

Table 1. Courses undertaken by First Year Students at the School of HBP

Semester 1	Description	Credit Hours	Semester 2	Description	Credit Hours
RUS 101	Integrated Studio 1	5	RUS 102 or RUS 103	Integrated Studio 2 or Design Studio	5
RAG 121	Environmental Science	3	RPG 131	Applied Statistics	3
RAG 132	Built Environment	3	REG 162	Design Structure	3
RMK152	Building Economic	3	RMK 252	Project Management	3
RAG 161	Building Construction	3	REG 264	Information Technology	3
Total		17	Total		17

Table 2. Structure of Degree Programs at the School of HBP

	High School Certificate, Matriculation, Diploma in Built Environment					
Year 1	Interdisciplinary Courses in Built Environment					
Year 2	Architecture	Interior Design	Urban & Regional Planning	Construction Management	Building Technology	Quantity Surveying
	Industrial Training [10 - 12 Weeks]					
Year 3	Architecture	Interior Design	Urban & Regional Planning	Construction Management	Building Technology	Quantity Surveying
	Bachelor Degree [B. Sc.]					
Year 4	Architecture [B. Arch.]	Housing	Urban & Regional Planning	Project Management	Building Technology	Landscape Architecture
Year 5	Architecture [B. Arch.]					Landscape Architecture
	Master Degrees [M. Sc.] Proceed to PhD					

construction covering the process of building construction from the beginning until completion. The students are introduced to normal building construction as well as the latest construction technologies. The subject of construction history has never been taught as a course by itself. However, it has been incorporated as part of the syllabus in other related courses such as building construction, structural engineering and measured drawing.

Findings of Construction History Survey

A survey on construction history was conducted on 328 HBP students to solicit their competency and perceptions on learning of construction history and facilities available at the School. Every effort was taken to ensure the student sample was picked at random. The profile of the student respondents is shown in Table 3. The sample of 328 students only represents 37.3% of a total of 879 students currently studying at the School.

In this survey, one of the questions asked was whether the students had taken courses that included construction history in their syllabus. From 16 courses listed in the questionnaire, 15 of them were voted as having construction history element in their syllabus, in which course RAG 132 - Introduction to Built Environment and Human Settlement [68.6%] and RAG 161 - Building Construction 1 [35.7%] were top of the students' response list. This might show that these two courses had incorporated construction history element in their syllabus compared to other courses. The students' response for others including courses listed down by the students themselves was rather low i.e. below 10%, which might imply that the elements of construction history in their syllabus were very minimum. Table 4 shows the results of students' response on the courses with construction history element.

Analysis of the importance of studying construction history elements namely evolution of construction, construction technology, use of materials as well as construction concept and design

Table 3. Profile of Student Respondents in Construction History Survey

Items	Response Categories	No. of Respondents	Percentage (%)
Gender	a. Male	138	42.1
	b. Female	190	57.9
		328	**100.0**
Major	a. Integrated	132	40.2
	b. Design-based (include Architecture, Interior Design, Urban Planning & Landscape)	89	27.1
	c. Non-design-based (include Project Management, Quantity Survey, Building Technology & Housing)	107	32.6
		328	**100.0**
Level	a. Undergraduate	304	92.7
	b. Post Graduate	24	7.3
		328	**100.0**
Year of Study	a. B. Sc. HBP		
	i. First Year	132	40.2
	ii. Second Year	78	23.8
	iii. Third Year	64	19.5
	b. B. Arch.		
	i. Fourth Year	10	3.1
	ii. Fifth Year	20	6.1
	c. M. Sc.	24	7.3
		328	**100.0**

Table 4. Students' Response on Courses with Construction History Elements

Code	Title of the course	Students' Response
RAG 132	Introduction to Built Environment and Human Settlement	225 [68.6%]
RAG 161	Building Construction 1	117 [35.7%]
RDG 334	Theory and History of Art Design	7 [2.1%]
RAG 232	Architectural Working Drawing and Documentation	6 [1.8%]
RAK 232	Principles of Architectural Design	26 [7.9%]
RAK 344	History and Theory of Architecture 1	17 [5.2%]
RAG 343	Housing Studies	30 [9.1%]
REG 361	Methods of Construction	11 [3.4%]
RPK 332	Urban Design	7 [2.1%]
RAG 265	Building Construction 2	26 [7.9%]
RPK 222	Conservation	8 [2.4%]
RAK 442	History and Theory of Architecture 2	9 [2.7%]
RAG 562	Building Technology	4 [1.2%]
RPK 535	Rural and Regional Planning	7 [2.1%]
REG 561	Construction and Materials Technology	6 [1.8%]
RET 561	Construction Technology	0 [0.0%]
Others		
RPK 231	Principles of Planning	9 [2.7%]
RPK 321	Landscape Planning	10 [3.0%]
RAL 371	Measured Drawing	3 [0.9%]

amongst students showed that most students had rated them as «Very Important» and «Important». These are shown in Table 5. The results show that majority of the students had agreed that these were significant elements in construction history studies.

Analysis of the levels of current facilities provided by the University and the School for courses with construction history elements showed that the students' rating fell mostly in «Good» and «Fair» categories as shown in Table 6. However, a higher percentage of students had rated that most of the current facilities provided as «Fair». This implies that these facilities are needed to be improved in the future.

As regards to the need of construction history as a new course at the School of HBP, 48.5% of the students had agreed that it was necessary to create this course while 31.4% disagreed and 20.1% were not sure and no answer. However, Table 7 shows that majority of the students [60.1%] had agreed that construction history was necessary in the built environment professional career.

Challenges of Construction History in Teaching and Learning in Built Environment

Based on the survey findings, there are several challenges facing the teaching and learning of

Table 5. Students' Rating of the Importance of Studying Construction History Elements

Construction history elements	Very Important	Important	Slightly Important	Not Important	Not Sure	No Answer	Total
Evolution of construction	100 [30.5%]	164 [50.0%]	40 [12.2%]	9 [2.7%]	4 [1.2%]	11 [3.4%]	328 [100%]
Construction technology	159 [48.5%]	127 [38.7%]	26 [7.9%]	2 [0.6%]	4 [1.2%]	10 [3.0%]	328 [100%]
Use of building material	142 [43.3%]	132 [40.2%]	32 [9.8%]	5 [1.5%]	4 [1.2%]	13 [4.0%]	328 [100%]
Concept and design of construction	177 [54.0%]	114 [34.8%]	18 [5.5%]	3 [0.9%]	2 [0.6%]	14 [4.3%]	328 [100%]

Table 6. Students' Rating of Facilities Provided by the University and the School of HBP for Courses with Construction History Element

Facilities	Excellent	Good	Fair	Poor	Bad	No Answer	Total
References at University's library	23 [7.0%]	132 [40.2%]	120 [36.6%]	27 [8.2%]	18 [5.5%]	8 [2.4%]	328 [100%]
References at HBP's Resource Centre	17 [5.2%]	99 [30.2%]	141 [43.0%]	48 [14.6%]	14 [4.3%]	9 [2.7%]	328 [100%]
Lecturers' presentation methods:							
Power point presentation	28 [8.5%]	104 [31.7%]	128 [39.0%]	38 [11.6%]	10 [3.0%]	20 [6.1%]	328 [100%]
Overhead projector (OHP)	23 [7.0%]	124 [37.8%]	127 [38.7%]	40 [12.2%]	8 [2.4%]	6 [1.8%]	328 [100%]
Opaque projector	8 [2.4%]	77 [23.5%]	130 [39.6%]	24 [7.3%]	13 [4.0%]	76 [23.2%]	328 [100%]

Table 7. Students' Rating of the Need of Construction History

Construction History Rating	Necessary	Not Necessary	Not Sure	No Answer	Total
As a new course at HBP	159 [48.5%]	103 [31.4%]	64 [19.5%]	2 [0.6%]	328 [100%]
Its significance in professional career	197 [60.1%]	71 [21.6%]	58 [17.7%]	2 [0.6%]	328 [100%]

construction history at the School of HBP. These challenges are discussed as follows:

- *Restructuring the syllabus of the built environment courses in tune with the construction history requirements.*

  One of the key challenges in the built environment education [as the case in most academic disciplines] is striking a balance between what is taught in academia with the needs and requirements of the construction and building industry. Even academics at the School of HBP who have been actively involved in collaborative researches and consultancies and have been elected as advisors to a number of professional and accreditation bodies oversee that such elusive balance is ascertained.

- *Sufficient training in building construction.*

  One of the recommendations posed by Ahmad, A.G. (2002) to prevent any historical buildings from being demolished or haphazard by modern development is by providing sufficient training and skills to the workers involved in preservation and conservation projects. In view of that, it is therefore, very important to re-assess our thinking and approach on heritage buildings conservation. The School of HBP should be more proactive in offering both formal courses and electives in the recording and documentation of historical buildings construction particularly in methods and techniques required for conservation works. HBP students seeking for practical training over the period of three months between year 2 and 3 should be encouraged to work at reputable firms involved in building construction industry.

- *Collaboration and cooperation between the University and the construction industry.*

  More concerted efforts should be undertaken to involve students to the real employment world. For instance, local authorities and other bodies should commission more projects to the University, particularly the School of HBP so that the students could be engaged in live projects in order to increase their knowledge and understanding of the opportunities, limits and constraints at construction works. Such collaboration between the University and the construction industry may also be beneficial in generating better mutual understanding to complement mutual needs.

- *Producing graduates who complement the needs of building conservation.*

  From the establishment of the School, HBP graduates have generally been well-accepted by employers in the construction and building industry. The relatively higher level of graduate employability may be due to the overall broad-based education in the built environment. It is anticipated that with the gained knowledge and competency in the fields of construction history and conservation, HBP graduates would not have any problems in getting employed in the field of building conservation. Nevertheless, there should be continuous efforts by the lecturers to vigorously focus on aspects of construction history in their teaching of the built environment courses.

CONCLUSIONS

The paper highlights the experiences of teaching and learning of construction history at the School of Housing, Building and Planning, Universiti Sains Malaysia, Penang, Malaysia. Findings from the students' survey have illustrated that construction history is far more needed in their studies and their professional undertakings in the built environment. Challenges that lie ahead of the School are also highlighted in terms of restructuring the syllabus of courses, training, collaboration and cooperation, and competence graduates. It is envisaged that with the full support from the University and the spirit of teamwork among the academic staff, the School of HBP is well-poised to overcome all these challenges and excel in producing promising graduates who are well-equipped with greater knowledge in construction history. This indirectly enables them to become future experts in construction history including preservation and conservation of heritage buildings. Therefore there is a need to teach construction history not only at the School of Housing Building and Planning, Universiti Sains

Malaysia, Penang but also at other higher learning institutions in Malaysia.

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Helical masonry vaulted staircase in Palladio and Vignola's architectures

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Helical masonry vaulted staircases are complex structural elements because of their geometry, technology and mechanical behaviour. These elements have been found in some historical buildings, and their *Firmitas* and *Venustas* wake the designer up nowadays.

The research aim is to analyse the masonry staircases built during the second half of '500, and in particularly these built by eminent architects and writers of treatise as Andrea di Pietro Dalla Gondola, called Palladio, and Jacopo Barozzi da Vignola.

The research of historical bibliographic sources is important and fundamental for investigating any historical architecture. Therefore, the architectural, technological and structural aspects have been considered simultaneously to analyse the structural behaviour of helical masonry vaulted staircases. The author have consulted treatises and handbooks of XVI c. and following, besides observing and surveying these element in situ.

The detailed analysis of these different aspects allows to formulate a static scheme of the structure, boundary conditions and the applied loading. Some architectural elements seem to have a main role in the staircase Statics, guaranteeing its structural behaviour in service conditions.

Introduction

The staircase has always been an important architectural element, both for symbolic meanings related to it and the difficulties due to its design and building. The staircase is a fascinating design theme for artists. It is a comparing standard, perhaps a way to compare themselves in terms of ability. During the Humanism, famous artists as Leonardo and Francesco di Giorgio Martini designed staircases full of symbolic meanings, whereas Alberti, in his treatise (Alberti 1485), emphasises that staircase is an architectural element which is difficult to be realised. The first monumental staircases date back to the beginning of XVI c. due to Michelangelo, Bramante, Antonio da Sangallo and others. Subsequently, the theme is investigated deeply and with curiosity as a «challenge», until the Seicento. During this century the staircase theme achieves the maximum inventiveness.

The staircase theme in historical architecture is extremely wide and complex, although the referring bibliography is limited. Therefore, the authors have turned their attention to the topic referring to a precise historical period, the second half of '500, and to two architects which are the most important in Northern Italy, Palladio and Vignola, limiting the research to only one structural typology, the helical masonry vaulted staircase.

The staircases in treatises

Since XV at XVI century

The first treatises of Architecture divulge during the XV and XVI c. During this period, the «pubblicistica»

spread everywhere. Indication of staircase design, comfort, safety and collocation in building could be found in every Renaissance treatise of Architecture. Only the last Renaissance treatise includes indications of building procedures: Scamozzi's treatise at the beginning of XVII century.

The *Vitruvio*'s treatise is the reference for architects during the Humanism and the Renaissance (Gambardella, 1993). However, the staircases are not taken into much consideration in this document; in fact, as Frà Giocondo Da Verona complained in edition of 1590, the rules for staircases design are missing. This is due to the fact that the staircase is not and architectonical element in the roman *domus*, whereas it is an imposing base in the public buildings, and often it becomes an architecture itself, as Serlio said (Serlio 1584).

Already in «De Re Aedificatoria» of *Leon Battista Alberti* (Alberti 1485), the staircase has an important role as useful element in the building but, at the same time, its presence makes the design difficult. The staircase is so necessary that «who would not like the staircases hinder, he avoids hindering the staircases», underlining that it is not possible to neglect the place devoted to them, although this could induce some difficulties to the designer in term of space organisation.

In *Francesco di Giorgio Martini*'s treatise, titled «Architettura civile e militare», the winding staircase is represented (Gambardella, 1993). It is located in a tower, as only entrance to a second defensive tower; in this way, whereas the enemy cover the long climb, the soldiers organise the defence. The staircase becomes tool of war and life.

If, in *Cataneo*, the courtyard is the centre of building and the staircases regulate only the space organisation (Barozzi and Cataneo 1560), *Alvise Cornaro* in 1556 wrote (Barozzi and Cataneo 1560): «the designer has to give space to staircases and not hinder them, because hindered, they hinder», following the same principle of Alberti.

In *Sebastiano Serlio*'s treatise (Serlio 1584), «I sette libri dell'architettura», the author declares that the helical staircases are «arduous structures to be built so that who is not able to design the traditional ones he should not even try to design those are more complex». The staircase is considered a difficult element which is understandable only to expert designer. He inserts the staircase in some perspective studies and he tries to furnish an handbook for designer. He takes again the theme in volume III, dedicated to Roman antiquities, where he writes about the staircase —building: Colosseo. In the analysis of Roman architectures, Serlio emphasises well made staircases, examples have to be followed by the contemporary architects. The Bramante's helical staircase stands out among the quotations; it is the only modern architecture, which has to be admired and studied. It is pointed out that the stairwell is designed carefully in every plate; evidently, the architect considered the staircase an important and difficult element in the architecture.

Giorgio Vasari, in «Introduzione all'Architettura» having an informative, teaching and technical aim, furnishes brief and shorted description, but extremely important for understanding how this architectural element is considered at this time. The staircases used by people in public building have to be comfortable, not steep, roomy, bright; they have to be magnificent. Besides, the author recommends that the designer finds an appropriate place for stairacse because «this element is difficult to be located in the building» (Barozzi and Cataneo 1560). The staircase is almost more important than main rooms; it is a needed structural element having also representative role, like a beauty gauge of construction so that «a lot of people see the staircase and not the remainder of the building» (Barozzi and Cataneo 1560).

The staircases realised by eminent architects, as Bramante, Michelangelo and *Philbert De l'Orme*, are many. The last one gives a definition of staircase in his manuscript «Le troisième livre de l'architecture», 1567: it is like the «beating heart» of building (Gambardella 1993). The main example that has influenced many architects is the double staircase of *Chambord* (1515–1525), built by Domenico da Cortone for the will of Francesco I, probably following Leonardo da Vinci's project. In fact, *Vignola*, who has been in France for a long period, would be influenced greatly, whereas *Andrea Palladio* has drawn and described it, figure 1, in the first of his «I Quattro Libri dell'Architettura», with that of Bramante and some Roman antiquities. In Palladio's treatise, in the same chapter, many detailed figures and many indications could be found concerning the staircases position in the building; besides, he catalogues staircase typologies that could be built. Technical indications about their built are not in this treatise.

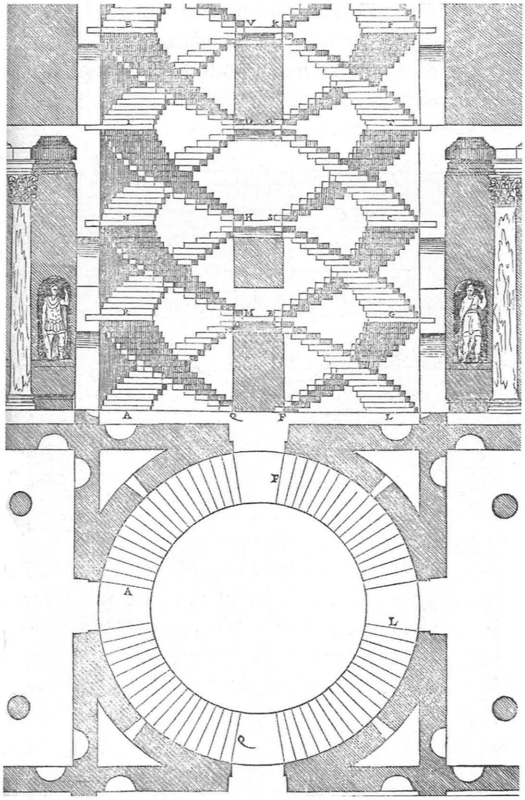

Figure 1
Staircase of Chambord in Palladio's drawing (Palladio 1570)

The first who gives this sort of indications is *Vincenzo Scamozzi*. In his treatise, «L'idea dell'architettura universale» (Scamozzi 1615), he is interested in architectural, technical and building aspects of staircases. He defines the right position of staircase in the building and different typologies in Libro II, the ornaments in Libro VI and the building indications in Libro VIII. He is the first who has distinguished the service staircases from the main ones explicitly. Besides, he indicated which material has to be preferred, in particular he disapproves wood due to its inflammability. The chapter about building indications is not clear and it is lacking in building details; however it is the most complete treatise up to this century.

The lack in building indications in treatises up to the XVII c., previously cited, is probably due to architect who relies on mason ability. The architect is interested in formal and figurative aspects, delegating the staircase realisation to the mason, who knows well materials, building technologies due to his builder's yard experience.

Modern centuries

It is possible to find more information about building technologies looking for in the treatises close to XIX c., when *Breymann* dedicated a book of his treatise to staircases (Breymann 1853). In this book, for the first time, a distinction of staircases is proposed in two categories: staircase «a collo» (it leans on continuum support, as piers, columns, walls, ecc.) and staircase «a volo» (it leans on support only at the beginning and the end of the flight), differing for building technology. This distinction has been used widely and it is also in *Daniele Donghi*'s handbook (Donghi 1923). These two treatises are still today the main references about technology in historic architectures.

Another distinction could be done in term of material for staircase building: stone staircase and masonry staircase. The manuscripts on stone staircases are numerous. The main reference is «Encyclopedie Medievale», wrote by *Eugéne Emanuel Viollet Le Duc*, where stone brick staircases, which are widespread in Northen Europe, are precisely described and drawn (Viollet Le Duc 1868). According to *Breymann*, the choice to build a masonry staircase, rather than a stone one, has simply an architectural reason because it is against any economical and practical aspiration. The professor of Stuttgart reports the Northen Europe building culture in his handbook. The clay brick has a secondary role due to plenty of stone and its traditional working, especially about its cutting. Instead, in country like Emilia Romagna, bricks are the main employed material for every buildings, due to the lack in stone and the plenty of clay. The difficulty related to bricks employment is undeniable, especially for vaulted structures which are built assembling many bricks compared to a monolithic beam. It is singular that *Palladio* works in a country where he could choose between clay brick and stone, therefore his choice to design a vaulted staircase has an architectural reason. In particular, according to *Donghi*, the safety of

masonry vaulted staircase, which is included in «a collo» staircase, depend on boundary continuity.

The vaulted masonry staircases could be built with the following technologies:

- *Staircase on vaulted beam:* is built on flat arch or plate-bande at each riser. In this way, only one wooden centering could be used, moving it following the staircase building. This technology is used for service staircases with small span, without architectural value; the vault intrados is covered with plastered «cannucciato» or similar.
- *Staircase on flying vault:* is built on flying vaults «a collo d'oca». The landings have to be built with particular care because they support the flights as abutments. The vault thickness is 14÷17 cm or more (multiring), depending on vault span. The staircases on flying vault could be symmetrical or asymmetrical, called respectively «a collo d'oca» or «zoppe».
- *Staircase on barrel vault:* Breymann says «the most simple staircases are those with linear flights and landings, that could be easily supported by barrel vaults» (Breymann 1853). However, it is possible to find some example in which the barrel vault has a curvilinear development as in figure 2.

Thick masonry walls need for this typology for carrying the abutments trust; the vault could be cylindrical or flat. The helical staircases on barrel vault are unusual for the complexity of their geometry and the accuracy needed for bricks disposition.

- *Staircase on Roman vault:* this typology of masonry vault is widespread in Southern Italy, especially at Rome. It is particularly important because it has been used by Vignola in his spectacular helical staircases. A first description of this typology could be found in Donghi's and others handbooks, figure 3. «A structural solution of great interest could be the cantilever staircase on roman vaults, made up of three flying vaults, two middle vaulted landings and one main vaulted landing; the vaults are made of clay bricks with small dimensions, flat or head disposed. The flying vaults look like special vaults «a collo d'oca», carried by boundary masonry walls and the adjacent vaulted landings, where middle landings are one quarter of vaulted rip roof and the main landing is a barrel vault» (Macchia and Oggioni 1995).

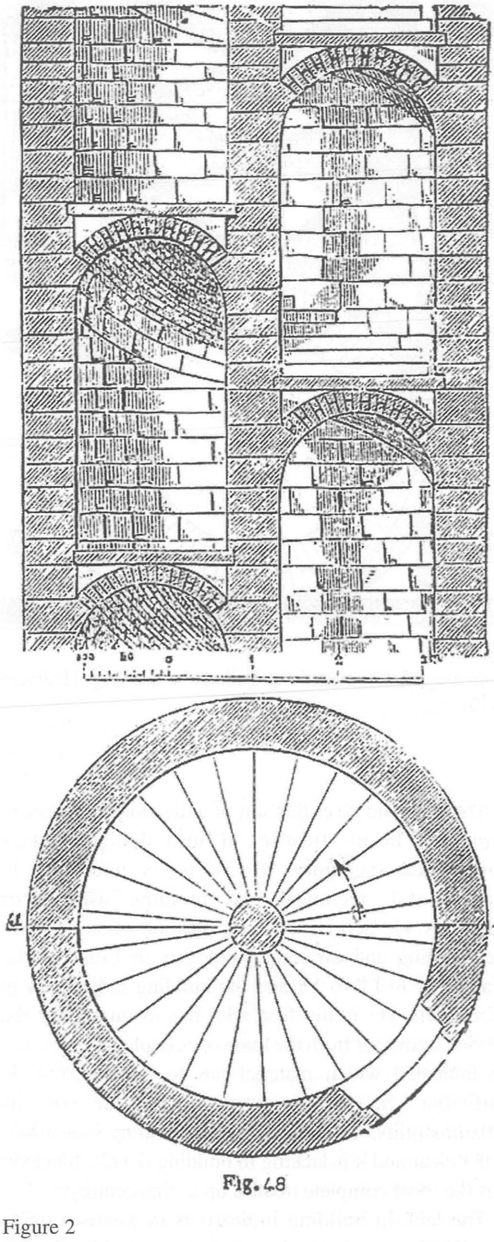

Figure 2
Helical masonry vaulted staircase (Breymann, 1853)

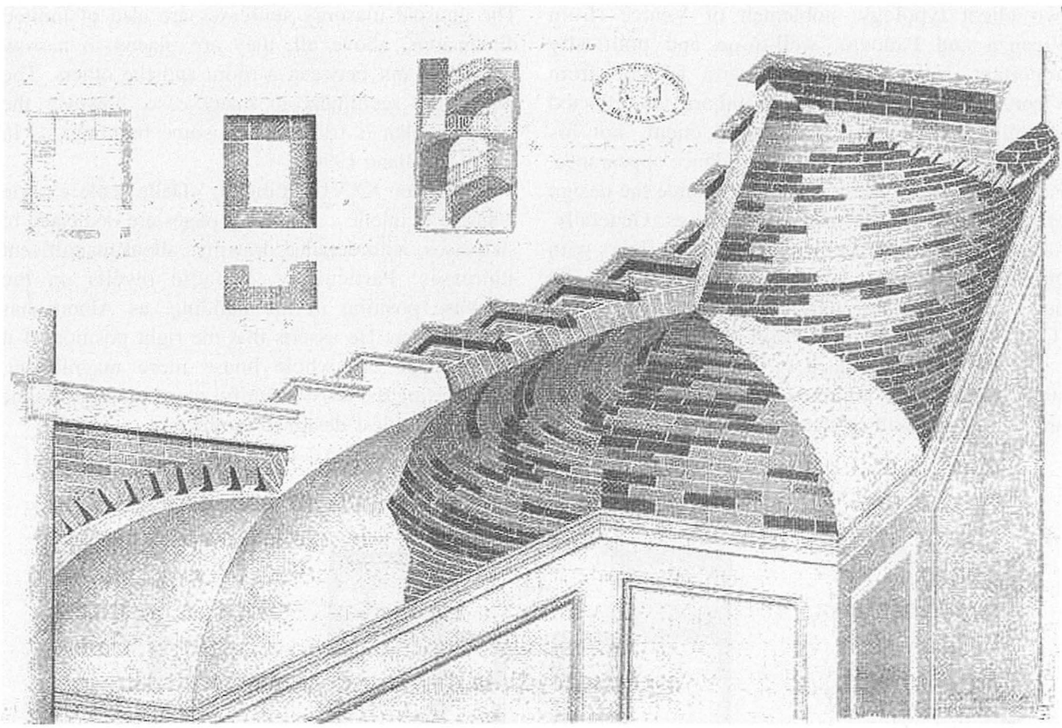

Figure 3
Staircase on Roman vault (Formenti 1909)

The particular disposition of bricks guarantees the Statics of vault, which looks like half arch. Besides, Vignola understood that, removing landing, the cantilever structure could be continuum and follow the climb without helical staircase breaks. The Vignola's helical staircase differs from the traditional one.

PALLADIO AND VIGNOLA COMPARED ABOUT STAIRCASE THEME

During the second half of XVI c., Andrea Palladio, probably the most famous architect, turns his attention to staircase theme, avoiding complex structures and hiding them in massive masonry walls. However, he realises, in villas, the only masonry vaulted staircases in Veneto, during this period. Contemporary in Emilia, Jacopo Barozzi da Vignola designs his wonderful staircases, which are architectural and technological masterpieces for their hazarded shapes obtained with a wise employment of materials and geometrical rules. The artists turns their attention to this theme in different way even if using the same materials and architectural shapes.

The staircase in Palladio

The role of staircase in Palladio's architecture (1508–1580) is a debate theme for many years. Palladio creates an architectural system which is described in his illustrated treatise «I Quattro Libri dell'Architettura», which is divulged as far as the Northern Europe. The authors analyse Palladio as villa designer, the building typology that has given him worldwide fame.

Veneto Villa is an architectural typology which is defined for the first time by Palladio around the middle of XVI c. in book II. Generally, Palladio has

two client typology: noblemen of Venice (from Vicenza and Padova), well-to-do and politically important, and noblemen of «terra ferma» (from Vicenza e Verona). The client authority is reflected in building: the social status of client, not his economical power, defines the residence appearance. Palladio indicates the criteria which guide the design of noblemen, lawyers, merchants houses. Generally, the villas for Venice noblemen have two floors with pronao at the front, whereas the others villas are developed in width with only one columns row (Conforti and Tuttle 2001; Puppi 1995).

The staircases designed by Palladio are made of stone and the stairs are fixed at the lateral walls; they are located in small rooms, far from the main rooms. The unusual masonry staircases are also of limited dimensions, above all, they are placed in narrow rooms cut out between a room and the others. The flights are rectilinear in many case, however the staircase plan is triangular in some buildings as in Rotonda (Blanc 1996).

In chapter XXVIII, Libro I, «Delle scale e varie maniere di quelle . . . » many pages are dedicated to staircases, with detailed drawings about magnificent staircases. Particularly, Palladio dwells on the staircase position in the building, as Alberti has already done. He asserts that the right position of it could make the whole house more magnificent, emphasising the most beautiful rooms and hiding the smallest and less designed rooms.

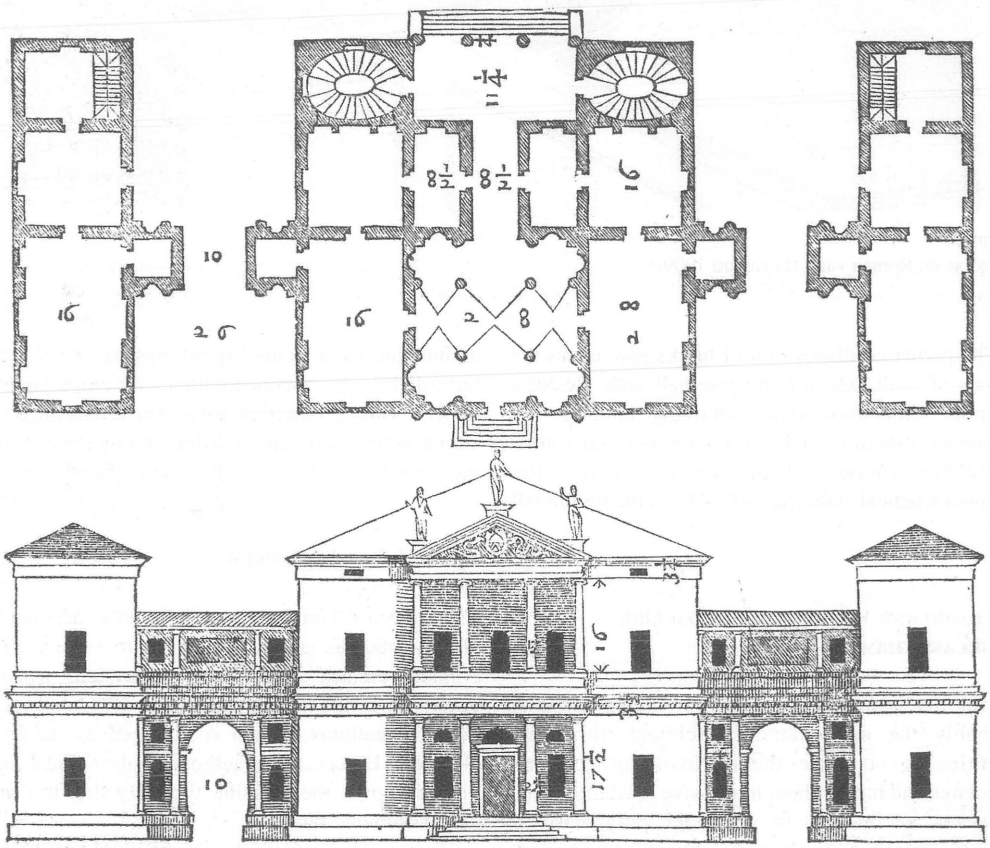

Figure 4
Plan and prospect of Pisani Villa (Palladio 1570.

Although these indications, it seems that Palladio does not practise his teachings: in many villas, the staircases are located in small and hidden rooms, difficult to be used. Also when the staircase has wide dimensions, they were enclosed in thick walls and they are not part of house architecture, because they were enclosed in specific rooms which are not visible from the main rooms, figure 4.

The staircase becomes a secondary element, necessary for enjoying the building but not for its architectural characterisation. Palladio does not employ the architectural code of this period, as concave and convex staircase, placing his staircases in small towers hidden in the load bearing structures. Palladio reduces the staircase to a joint element between one floor and the other. The staircase as architectural element appears only outside: magnificent base which raises the building from ground, recovering the staircase theme of religious architecture (Chastel 1965).

Although that, Palladio realises three particular staircases for architectural shape and technology. These staircases are located in three villas belonged to Venice noblemens from 1550 to 1560: villa Pisani, now Placco (1552), Montagnana (Pd); villa Cornaro, now Gable (1553), Piombino Dese (Pd); villa Foscari «La Malcontenta» (1559–60), Mira (Ve).

These buildings are the only that have helical masonry vaulted staircases. They are a particular typology of villa, following Ackermann's indication (De Fusco 1981), having two floors with two columns rows, isolated without rural buildings around. The villas in Montagnana and Mira are not exactly villas. The first one is called by numerous authors Palace due to the neighbourhood of Montagnana, the second one is realised as holidays palace close to Venice, without farm role. Pisani and Cornaro Villas have a four columns entrance, two helical staircases on the back for jointing the two superimposed loggias towards the garden (Prinz 1969).

The staircase in Vignola

The Jacopo Barozzi da Vignola's work (1507–1573) is characterised by staircase as fundamental element in architectural house composition. Loukomski said that Vignola uses «staircases, obelisks, frescos and porticos, placing them to do better views» (Loukomski 1927). The architect lives in Rome, Emilia and France. His young training is done as painter in Serlio's school, according to some researchers. During his training he works on architecture, sculpture and hydraulic engineering. Undoubtedly, Vignola is a careful observer of architectures realised by his contemporaries and he has carried out important architectural and structural starting points. In particular it is possible to find the influence of Baldassarre Peruzzi and Giulio Romano (Orazi 1982; Walcher Casotti 1960; Tuttle et al. 2002).

With the purpose to compare Palladio and Vignola about the same theme, the authors consider the staircase realised in private buildings, in particular in *Country Palace* which has different characteristics in Emilia than in Veneto villas. The country palace is designed and realised with the same criteria than the city palace; the palace is conceived like «a piece of city granted to countryside» (Cuppini and Matteucci, 1967) and its architecture is conceived apart from the adjacent agricultural buildings. The bigness and magnificence of building is proportional to the economical power of client, and not related to his social status (Scannavini 1998).

The staircases designed by Vignola are always spectacular and fruit of careful investigations. In fact, Vignola takes care of their design and dimension evaluation, choosing complex shapes and hazarded structural solutions. His staircases are calculated empirically without mathematical tools. It is observed from his drawings that one building cross section goes through the staircase always; in many project, there are different cross section of the same staircase or graphical analysis on the stairs dimension, as in Cervini Villa, figure 5.

The absolute knowledge of building technologies, materials, geometrical roles is evident in Vignola's masterpiece. Vignola knows that the staircase is a complex architectural element and, probably for this reason, he is fascinated by it.

The authors concentrate on the staircase of Boncompagni Palace, at Vignola (Mo). The palace attribution to Vignola is doubtful, whereas the staircase design looks to be his hand; instead, the realization of masterpiece is due probably to Bartolomeo Tristano (Tutle at al. 2002). The author reaches at this project, perfect synthesis of shape and dimension, after long work experience. A chronology

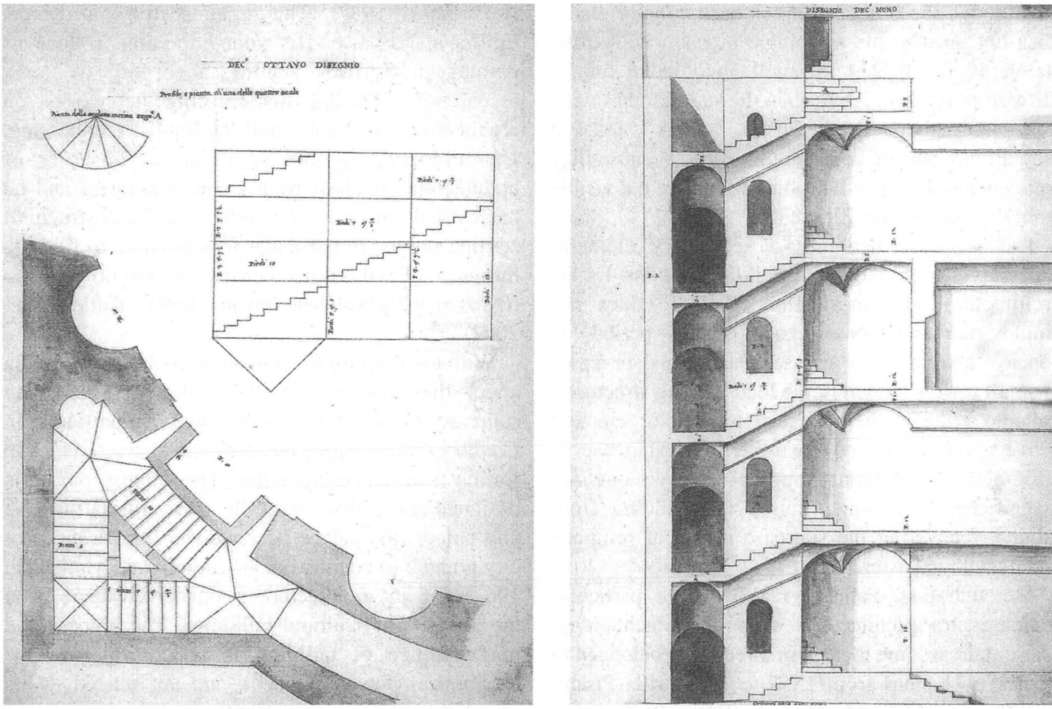

Figure 5
Graphic analysis of staircase in Cervini Palace (Tuttle et al. 2002)

of Vignola's staircases is reported in table 1. It could be not complete and exhaustive from historical and architectural point of view, but it is useful for demonstrating the experience path followed by Vignola.

Comparison on architectural, technological and building points of view

Palladio and Vignola turn their attention to indoor staircases in different ways. In Vignola the staircase is a spectacular element for its architecture and structure, figure 6, whereas in Palladio it is hidden in massive masonry walls, figure 7.

Really, both of them contain the staircases in turrets, defiladed respect to the others rooms, and prefer elliptical plan (Blanc 1996). While Palladio's staircases have a utilitarian function, to joint different floors, Vignola's staircases have a aesthetic function in building architecture and the whole staircase is seen by the user. Nevertheless, an aesthetic purpose could be found also in Palladio. Who covers the staircase leaves back a room, then he appears suddenly at another room, which is different or analogous, for looking at it from a different point of view. The staircase is the tool for showing the most beautiful parts of house.

Both the authors design unusual structures for shape and structural behaviour. From the technological point of view, either Palladio or Vignola employed clay brick masonry for building their masterpieces. This material has a low tensile strength and good compressive strength. Palladio chooses a material employed for realising structural elements which are usually compressed, rather than using monolithic stone blocks fixed at one or both ends. Besides, the masonry choice entails the design of a vaulted structure, which looks to be itself bearing for the bricks disposition, as in Pisani and Cornaro villas. The vault span is limited (about 1.60–1.70 m) and it is fixed to boundary walls and central pier.

Table 1. Chronology of Vignola's staircases

Time	Masterpiece	Typology
1530–1540	Isolani Palace (Minerbio)	Wooden helical staircase
	Boncompagni Palace (Bologna)	Masonry helical staircase
	Cervini Villa (Montepulciano)	Masonry vaulted staircase on triangular plan
1550–1560	Ninfeo in Giulia Villa (Rome)	Stone staircases
	Farnese Palace (Caprarola)	Masonry vaulted staircase «a collo» with columns support on one side
1560–1568	Farnese Palace (Piacenza)	Small helical staircases and traditional ones
	Isolani Palace (Bologna)	Masonry helical vaulted staircase with roman vault boundary conditions
	Boncompagni Palace (Vignola)	Masonry helical vaulted staircase with roman vault along the flights without landing
After 1568	Caprarola (hospital, Paziello Palace, playing court)	Numerous helical staircases
	Fortification walls of town for Savelli family (Castelgandolfo)	A magnificent and two small masonry vaulted staircases enclosed in fortification walls
	Town Hall of Castro (Viterbo)	Stone helical staircase (outlived the building after earthquake).

Instead in Foscari villa, the structure is composed of a system of flat arches, like plate-bandes, under each stair, jointed by flat bricks of stairs. The intrados is realised with plastered «cannucciato» vault shaped.

Vignola, who is used to build with masonry because in Emilia the stone is an unusual materials, distinguishes himself inventing a daring building technology for realising scenic and spectacular structures, able to resist static loading (dead load and service load), and also seismic loading (helical staircase in Town Hall of Castro). A particular technology is noted in two staircases: the first in Isolani Palace, at Bologna, and the second in Boncompagni Palace, at Vignola. These structures are designed starting from the roman staircase typology, modifying some building aspects. In fact, Vignola eliminates the middle landings designing a single continuum helical flight. In Isolani Palace, the staircase starts from the underground floor with a barrel vault fixed at boundary walls and at central pier; from the ground floor, the vault is configured as roman vault and there is not the central support, leaving place to stair-well and masonry parapet. The flight stops at the only final landing, which is realised like a barrel vault following the traditional roman staircase typology. The staircase in Boncompagni Palace seems to be an evolution of the first one. In fact, there is not final landing and the staircase is a continuum helical flight with roman vault.

Comparing both architects about materials for realising their staircases, it is evident that the mortar is different, although both of them used masonry. Palladio uses lime mortar, come from Veneto quarries. Vignola uses limes from Emilia, mixed with plaster (Marinelli and Scarpellini 1992). The plaster modifies the mortar characteristics giving expansive property; so that the bricks are compressed after the mortar curing.

From the technical point of view, meaning the quantity evaluation of mechanical forces, both the architects calculate structure dimensions with geometrical tools. The staircase realisation is the fruit of experiences carried out in situ and observing the past architectures. On the other hand they have not

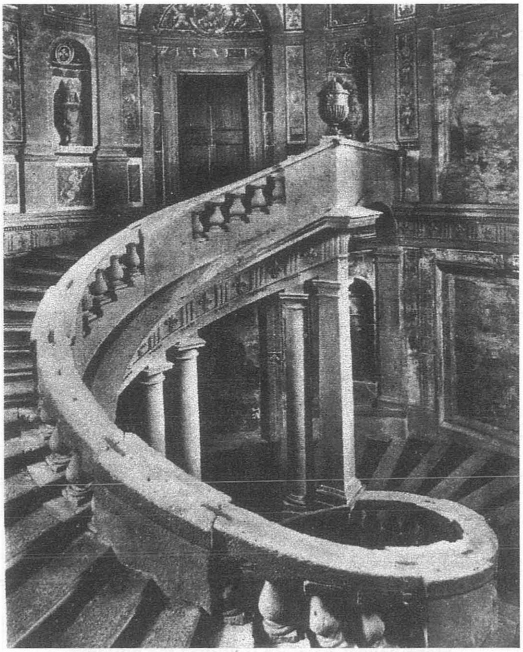

Figure 6
Staircase of Farnese Palace, Caprarola (Loukomski 1927)

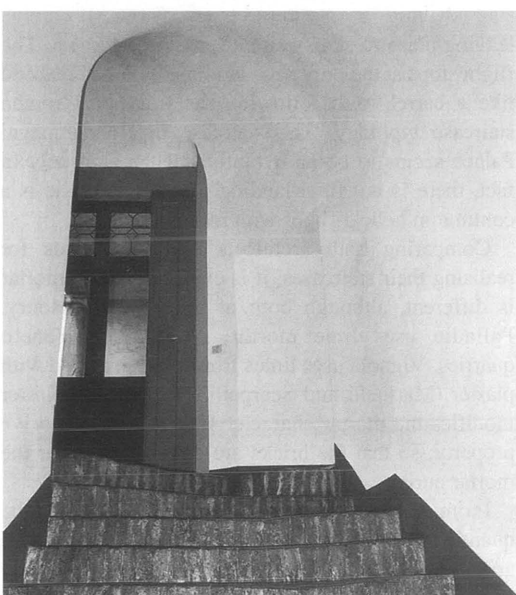

Figure 7
Staircase in Pisani Villa (Massarotto 2002)

scientific knowledge about material strength and structural behaviour of architectural elements.

STRUCTURAL BEHAVIOUR OF MASONRY VAULTED STAIRCASES IN PALLADIO AND VIGNOLA

The following remarks are based on direct observation of these structures, recording their geometry, masonry configuration and damages that have been in the past or it is going on. The structure considered in this research have not shown any damage, visible or historically documented. Staircases of Pisani villa and Boncompagni Palace are reported as examples of Palladio and Vignola respectively.

Palladio in Pisani Villa, Montagnana (Pd)

The staircase amounts to a particular vaulted structure, which is developed rotating a circular arch around the central pier; the plan is elliptical and the boundary walls are massive. It is plausible that the vaulted structure and boundary walls are realised contemporary. The vault intrados is almost everywhere

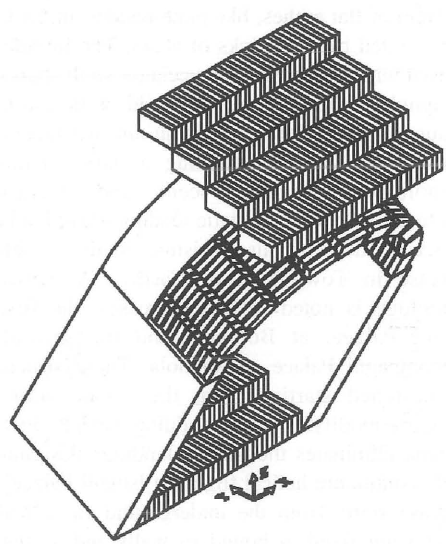

Figure 8
Tri-dimensional cross section of Pisani Villa staircase

like a round arch, besides some variations of curvature could induce to think that the vault has been realised without wooden centering. The bricks are tilted respect to the vertical direction toward the flight starting, about an angle so that the following ring leans on the previous one, figure 8.

The boundary conditions look to be fixed ends. The modest dimensions of these vaulted structure (span of 170 cm, thickness at crown 15 cm) suggest two different static scheme, figure 9: beam with variable cross section or round arch, both fixed at ends.

The analysis carried out for both of them has done for a generic stair (depth of 34 cm), loaded by a uniform dead load. A linear elastic analysis is carried on because the structure is not damaged. The stresses evaluated for the two configuration are less than the ultimate masonry strength, table 2, considering a ultimate tensile stress of 0.20 N/mm^2 (Belluzzi O. 1994). The fixed arch scheme is the best to describe the structural behaviour of this typology of staircase. This remark is supported by the bricks disposition, the massive walls and the central pier which are able to receive the horizontal thrust at springings, and finally by the calculated stresses.

Vignola in Boncompagni Palace, Vignola (Mo)

The structural behaviour of this staircase is more complex and it could not be reduced to a bi-dimensional problem, because the structure amounts to an helical flight fixed at the boundary walls, with almost circular plan, and collaborating with the parapet which is helical and has a thin rectangular cross section.

The flight cross section is variable from a minimum of 15 cm at fixed joint and a maximum of 30 cm at the other end, toward the stair-well; it looks as a rib flat arch with a span of 200 cm. Some steel bars have been found along the flight, one each three/four stairs, and they are closed to the intrados. The roman masonry vault appears like a series of half skewed arches fixed at the crown and free at the springing; the brick layers are tangent to the boundary walls and perpendicular to the stair-well. The masonry parapet is jointed to the staircase structure; it is 100 cm high and 15 cm thick. The whole structure is fixed at the flight starting and hinged at the flight end. In fact, the flight that connects the underground and ground floors amounts to a masonry barrel vault supported by boundary walls and central pier: the system seems to be like staircase foundation. The flight structure is an helical vault which rises for two floors (about 12 m), doing 720° (2 turns) for rising from a floor and the other, figure 10.

The static scheme able to describe staircase structural behaviour has to consider the whole structure, that is the collaboration between roman vaulted flight, parapet and boundary masonry walls. It is possible to give different interpretations.

Firstly, a single stair is considered. Its static scheme is a cantilever with variable cross section, fixed at the boundary walls and elastically hinged at the other end (rotational and extensional hinges),

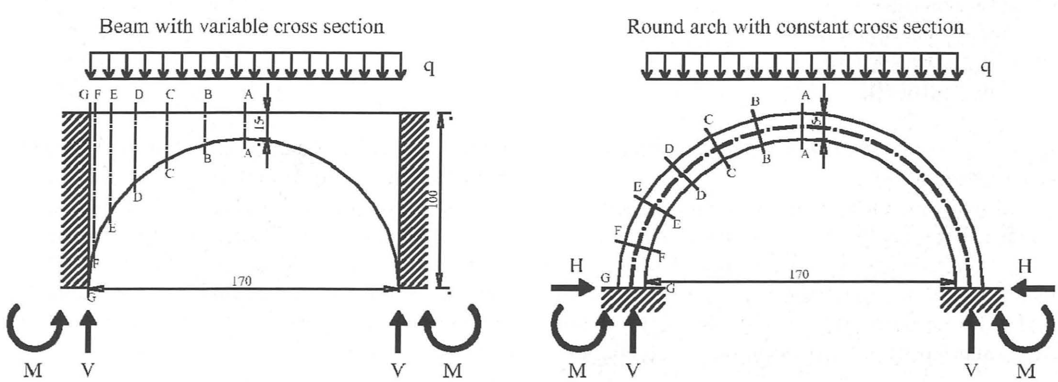

Figure 9
Static scheme for Pisani Villa staircase

Table 2. Stresses in Pisani Villa staircase, considering fixed round arch and fixed beam static schemes

Palladio's helical masonry vaulted staircase as fixed beam			
x [mm]	M [$N \cdot$ mm]	J [mm^4]	σ [N/mm^2]
0.00 10^1	–4.82 10^5	2.83 10^{10}	–0.0085
2.90 10^1	–4.33 10^5	2.59 10^{10}	–0.0081
11.39 10^1	–3.01 10^5	1.97 10^{10}	–0.0068
24.90 10^1	–1.20 10^5	1.20 10^{10}	–0.0038
42.50 10^1	0.60 10^5	5.39 10^9	0.0032
63.00 10^1	1.92 10^5	1.44 10^9	0.0248
85.00 10^1	2.41 10^5	9.56 10^7	0.1889
Palladio's helical masonry vaulted staircase as fixed arch			
ϕ [deg]	M [N·mm]	N [N]	σ_e/σ_i [N/mm^2]
0	1497 10^2	–267.96 10^1	–0.118/–0.249
10	232.31 10^2	–253.22 10^1	0.034/–0.091
20	–454.67 10^2	–232.58 10^1	0.112/–0.001
30	–690.88 10^2	–208.41 10^1	0.136/0.034
40	–608.01 10^2	–183.08 10^1	0.119/0.030
50	–334.07 10^2	–158.83 10^1	0.080/0.002
60	14.27 10^2	–137.67 10^1	0.032/–0.036
70	338.57 10^2	–121.29 10^1	–0.012/–0.071
80	564.21 10^2	–110.94 10^1	–0.042/–0.096
90	644.69 10^2	–107.40 10^1	–0.053/–0.105

σ_e stress at extrados.
σ_i stress at intrados.
0° cross section at springing.
90° cross section at crown.

figure 11, for simulating the parapet influence, as helical beam.

The analysis of a tri-dimensional structure is reduced to the analysis of many mono-dimensional ones. Therefore, the parapet works like an helical beam, fixed at the foundation and hinged at the top, loaded by dead weight and part of flight weight. The remaining flight weight is borne by boundary walls.

Secondly, the flight is evaluated as tri-dimensional element. The structure is schematised as an helical beam, fixed at the foundation and at the top, which cross section amounts to masonry vault and parapet, forming a complex cross section. The fixed hinges along the boundary wall could be neglected because the bricks of each arch are tangent it. The analytical solution of this static scheme could be found in bibliography (Belluzzi O. 1994, Pozzati P. 1972); the difficulty consists to define some geometrical characteristics (torsion inertia moment).

These static schemes are the starting point for developing the research on this staircase typology.

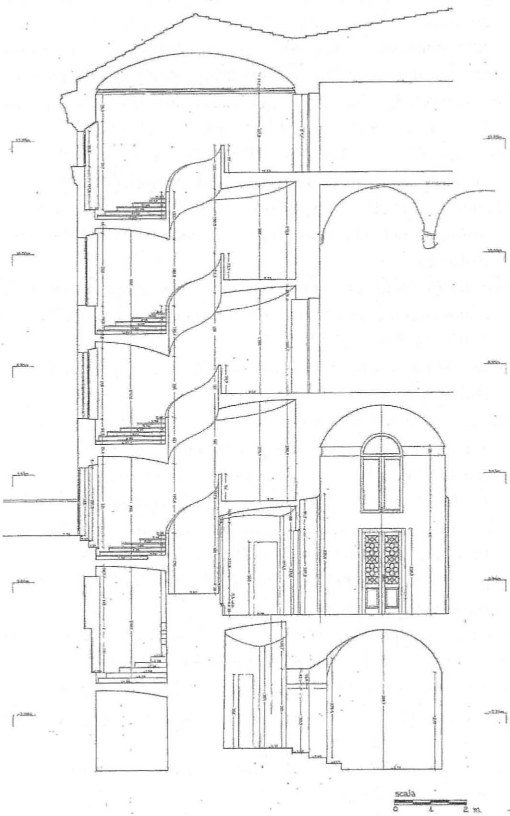

Figure 10
Cross section of Boncompagni Palace staircase (Lenzi and Ventura 2000)

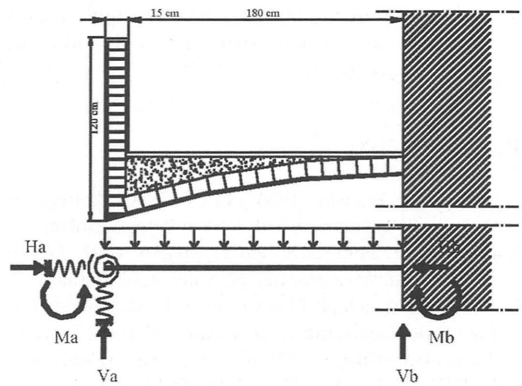

Figure 11
First static scheme for Boncompagni Palace staircase

The model complexity requires the employment of calculus tools as finite element method. The 3D modeling of Vignola's staircase is in progress.

Vignola's staircases are an unique masterpieces in the staircase architecture everywhere, which could not be catalogued within traditional technological and building definitions of handbooks.

Conclusions

Palladio e Vignola turned their attention to staircase theme, in private building, with different approaches. Concerning the helical masonry vaulted staircase, the following remarks could be done:

- Palladio's staircase offer a limited view due to the presence of central masonry pier, whereas Vignola's staircase is visible wholly with scenic views due to stair-well;
- both the architects enclosed their staircases in massive masonry rooms, choosing the best technology for them realisation;
- the structural analysis of these staircases has to be carried on considering the geometrical and technological aspects, as the historical one;
- their structural behaviour is difficult to be defined and a finite element model has to be done for verifying some hypotheses carried out during the research;
- their dynamic behaviour has to be investigated too. The Italian code imposes that the cantilever masonry staircases have to be demolished and rebuilt in reinforced concrete or steel (D.M. 02/07/81; D.M. 24/01/86; Lenza and Rampolla 1987); the restoration is possible only for historical and architectural value. Nevertheless, it is possible to preserve these structures defining their safety margin after a precise investigation about their geometry, technology and damage using calculus tools calibrated for the specific historical building (Barbieri, Foraboschi and Siviero, 1997).
- The author wish that the historical structures could be not replaced with new reinforced concrete or steel elements. Nowadays, it is possible to employ innovative materials, like

fiber reinforced polymers, able to supply tensile strength to masonry structures under ultimate loading condition.

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Italian's «forms» and local masonry in early French Renaissance: The stone coffered ceilings called «Voutes-plates», from the castle of Gaillon to the Bouton chapel in Beaune

Flaminia Bardati

Towards the end of the 15th century appear in France the first buildings inspired by the Italian Renaissance. Before the experience of Fontainebleau (around 1540's), where important artists such as Rosso Fiorentino, Francesco Primaticcio, Sebastiano Serlio and Jacopo Barozzi da Vignola will be working, Italian artists active in France are generally relegated to an advisory role, as is the case of Fra Giocondo in the reconstruction of the bridge of Paris, (1499–1500) or to works of a decorative character, in the case of Domenico da Cortona.

These artists, originating from the court of Naples, followed the king Charles VIII in France in 1495 (Montaiglon 1851–52; Fillon 1851–52, Lesueur 1929; Ciotta 1985). Sculptors like Guido Mazzoni, Antonio Giusti, Pace Gagini, Lorenzo da Mugiano, painters like Andrea Solario and Girolamo Torniello, and cabinetmakers like Riccardo da Carpi, contribute to the diffusion in France of the stylistic and cultural innovations elaborated in Italy during the 15th century. Under this influence French *maîtres-maçons*, trained in the masonry tradition of the cathedrals' building yards, create new «forms» inspired by the Italian Renaissance but executed by French hands. The new architectural language, stimulating the comparison and the blending of two different cultures, leads to the invention of new types of decoration and forces an adaptation of building techniques to the new aesthetic and cultural requirements. All these factors assume particular evidence in the domain of the arched covers.

The evolution of the French technique of stone construction has been analysed in full detail by Pérouse de Montclos ([1982] 2001). He has also focused on the numerous and particular shapes that the arched cover has been given in France in the 16th–18th centuries. My research is based on his findings.

The survey begins with the analysis of a new type of cover devised by French builders which intersect the gasket-vault and the panelled wood ceiling.[1] By observing a series of vaults realized in the north of France during the first half of the 16th century, it is possible to notice that, besides a generic desire to imitate the formal solution of the Italian covers, considered to be at the artistic vanguard, the motive of the coffer has a remarkable fortune, since it lends itself very well to be repeated as the recurring element in a regularly decorated field, apt to satisfy the French bent for ornaments.

The adoption of similar shapes collides with the traditional use of the rib-groined vault, characteristic of the *flamboyant* architecture of that period. The structural solutions reached in the ancient regions of Normandy and Poitou had solved the problem by stretching the constructive logic of gothic architecture to its extreme consequences.

The example just mentioned above, part of a wider research on the new types of covers realized with traditional techniques and materials, points to several solutions: from the great basket-handle vault with orthogonal branches and stone flags (*voûte en*

berceau en anse-de-panier à dalles sur nervures) of the castle of Chambord, particularly diffused in Touraine and Val de Loire,[2] to the flat stone ceilings supported by a series of small arches, characteristics of the castle galleries of the Center, the Poitou and the neighbouring regions, but used also to cover aristocratic chapels,[3] and finally, to the so-called *voûtes-plates dallées*, diffused especially in Normandy.

There is no exact translation of *voûtes-plates dallées*; first of all because they are not real vaults. They are a flat stone cover, resting on a supporting structure, generally part of a rib-groined vault, with branches and diagonal ribs, called *arc-diaphragm*, namely screen-arc. These are the ribs of a pointed vault, from which the groins have been eliminated and in which a parting wall supports the flat cover. Therefore, the screen is the parting wall that discharges the cover's weight on to the arches. Sometimes this parting wall can be completely fret-worked and reduced into a series of ribbings or little columns which channel the forces and distribute them on the arches.

This type of cover is completely different from the straight vaults based on the system of the flat arch, (*voûtes-plates clavées*), used already in the 12st-13st centuries in France (Reveyron, 1993), since the stone ceiling is supported by the *arc-diaphragm* structure, considered as a trilithon, without offering any static collaboration (Pérouse de Montclos, [1982] 2001, 162–163; Pérouse de Montclos 1989, 274). The covers of the Center and the Poitou are totally classifiable like *voûtes plates dallées*, even if the formal results are completely different from those of Normandy. In the case of square or rectangular spaces, characterized by short dimensions (up to 5 meters), the *arc-diaphragm* becomes ribbings, which form a series of supporting segmental arches, disposed orthogonally to the bearing walls. The stone slabs of the ceiling rest on this thrusting system. The presence of more segmented arches, placed parallel to the bearing walls, serves only a decorative purpose, to create regular ornamental fields, similar to Renaissance wood coffers. This type of cover characterizes the galleries of the castles of La Rochefoucauld (1518–1533), of Dampierre-sur-Bouton (half of 16th century), of the hôtel d' Escoville in Caen (1532–15409, of the western porch of the castle of Chambord (from 1540). It is also used in the stairs of the castles of Azay-le-Rideau (1515–1518), Poncé-sur-Loir (1542) and of the hôtel de Pincé in Angers (from 1523). In the case of galleries and arcades, often some greater arches are extended to cover all the clear passage of the gallery, subdividing the space into regular spans.

In all these cases, even if the buildings present some Renaissance ornaments, the covers still respond to the gothic construction logic, entrusting the bearing role to the ribs and discharging completely the stone surfaces, which are left only with a decorative function. Viollet-le-Duc does not hesitate, in fact, to place them in the tradition of French construction history, as the extreme expression of the gothic constructive system:

> Les Normands, les Manceaux, les Bretons, firent volontiers des voûtes composées, soit de grandes dalles appareillées, décorées de moulures à l'intérieur, se soutenant par leur coupes, sans le secours des arcs, soit de plafonds de pierre posés sur des arcs [. . .] Le système des voûtes gothiques devait en venir là, c'était nécessairement sa dernière expression. Fermer les les intervalles laissés entre les arcs par des plafonds, et, au besoin, multiplier les arcs à ce point de n'avoir plus entre eux que des surfaces pouvant être facilement remplies par une ou deux dalles, c'était arriver à la limite du système (Viollet-le-Duc, [1854–1868] 1997, 4: 122–124).

In this instance my analysis will focus on the *voûtes-plates dallées* which use the branches of a rib-groined vault, and on the evolution of this type of cover in Normandy during the first half of the 16th century.

The first two Norman buildings employing the *voûtes-plates dallées* are the porch of the Saint-Etienne-le-Vieux church in Caen and the lower chapel of the castle of Gaillon. These two covers can be considered the prototypes of the structure that I propose to analyse. In both cases the buildings are in bad condition and it is very difficult to visit them. In 1944, the church of Saint-Etienne had been strongly damaged during the six months of bombardments after the landing in Normandy and no repairs are envisaged yet; the castle of Gaillon, stripped of all its sculptures during the French revolution, has been transformed in jail in the first years of the 19th century and restoration work has started just recently.[4]

The lower chapel of Gaillon was constructed, most probably, by the *maître-maçon* Guillaume Senault,

already active in the castle of Amboise, in Touraine. In 1504 Richard Jouy provides some centerings for the cover (Deville, 1850, 109); we ignore whether the larger and magnificent upper chapel had the same type of cover (Huard 1926, 26; Pérouse de Montclos [1982] 2001, 162) or a traditional rib-groined vault (Bardati 2002, 135–139).

The lower chapel has a nave with a five sides apse. An external gallery surrounds the building and continues along the northeast side of the castle, constituting an exterior ambulatory, accessible from the north flank of the chapel. The nave and the ambulatory are covered by the *voutes-plates dallées* system. The *arcs-diaphragm* present a full parting wall, whose horizontal mortar beds totally correspond to those of the boundary walls of the chapel, Figure 1.

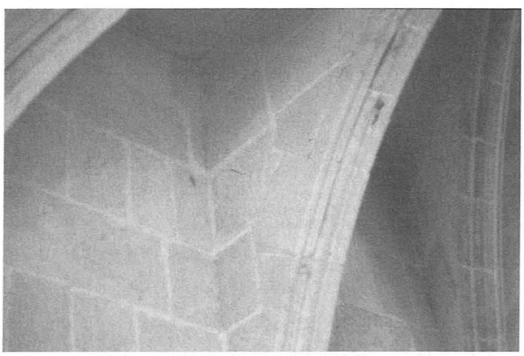

Figure 1
Castle of Gaillon (1498–1510). View of an *arc-diaphragm* of the lower chapel

Figure 2
Castle of Gaillon (1498–1510). A sculpted panel from the lapidary warehouse

Above the horizontal slabs rests the wood frame of the first floor deckhead, as we can see in the ambulatory, where some stone flags have been lost. Instead of what is written in the numerous descriptions of the castle of Gaillon (Bardati, 2002, 119–120) the lower chapel has no decorations, circumstance that does not help to understand the presence of the *voûtes-plates dallées*. A hypothesis attributes this absence to the vicissitudes of the castle during the French Revolution and to the consequent loss of the greater part of the ornamental elements: in fact, the fine carved stone fragment conserved in the lapidary warehouse of the castle could come just from the cited ambulatory, Figure 2.

This panel , with its quadrangular fields bordered by ribs and decorated with bas-relief, seems to reproduce a coffered wood ceiling, not dissimilar perhaps from those that in the same years Riccardo da Carpi was executing in the contiguous rooms of the Grand' Maison.

The cover of the porch of Saint-Etienne-le-Vieux introduces different characters. Realized between the end of the 15th century and the beginning of 16th century (Mancel 1846; CAF 1908, 1: 93–105), the porch, Figures 3, occupies the place of the fourth chapel on the northern flank of the church.

The entire structure is the expression of the *flamboyant* style: from the spire decorated with liernes to the structure of the cover, Figures 4, 5, 6.

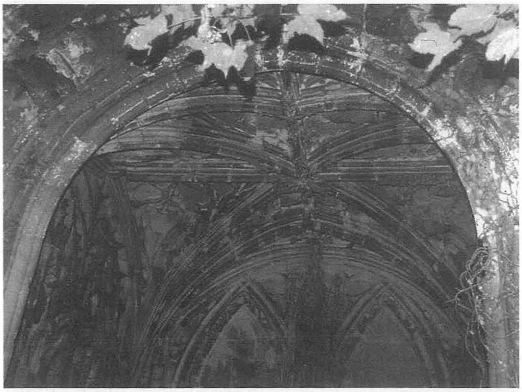

Figure 3
Church of Saint-Etienne-le-Vieux in Caen. View of the porch

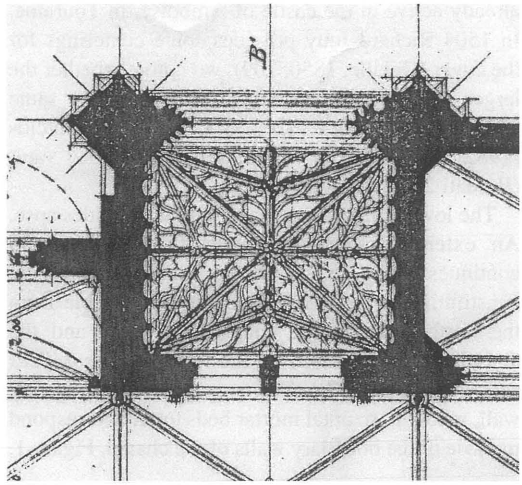

Figure 5
Church of Saint-Etienne-le-Vieux in Caen. Plan of the Porch's ceiling, executed by Brunet in 1901

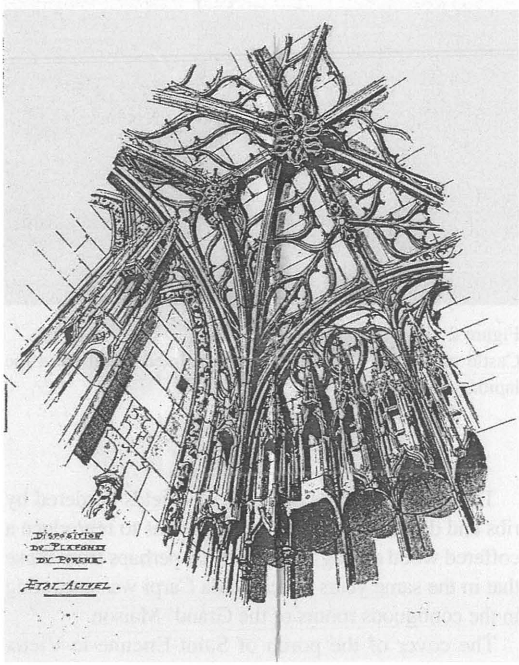

Figure 4
Church of Saint-Etienne-le-Vieux in Caen. Perspective view of the porch, executed by Brunet in 1901

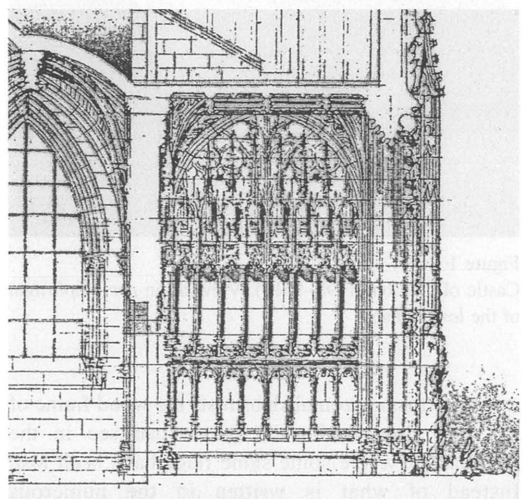

Figure 6
Church of Saint-Etienne-le-Vieux in Caen. Section of the porch, executed by Brunet in 1901

Here, in fact, the *arcs-diaphragm* are completely fret-worked: that means that in the place of the parting wall there are some sinuous ribs that connect the arches to the inner surface of the ceiling, Figures 7.

The structural system seems to go back to what had been realized in the first years of the 14th century in

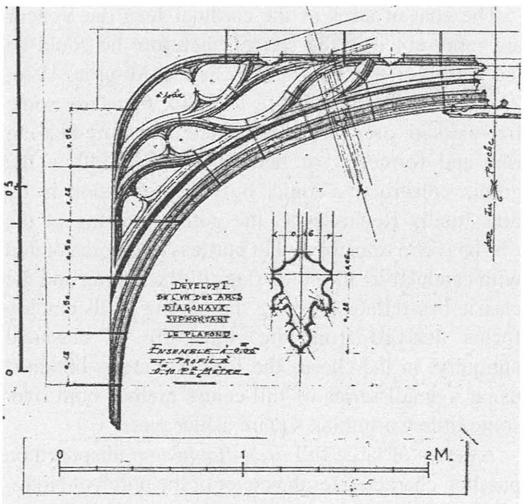

Figure 7
Church of Saint-Etienne-le-Vieux in Caen. Detail of the arc-diaphragm

England, in the cathedral of Bristol (1311–1332) and in the monastery of Southwell (1320–1330), where these fret-worked *arcs-diaphragm*, also defined «archi volanti» i.e. flying arches (Franchetti Pardo, [1997] 2001, 384) appear. In the church of Bristol, «the vault thrusts are brought down and equalized by a singular system of cross arches in the side aisles, supporting curious double vaults set sideways» (Harvey, [1950] 1974, 165).

In the Berkeley Chapel of the same church, only the ribs of the rib-groined vault remain, completely deprived of any groin, parting walls or fret-works. As observed by Franchetti, in this kind of vault the static component is used «come pretesto per attingere a risultati di ordine essenzialmente figurativo [. . .] forme derivate da elementi del lessico statico vengono spesso impiegate in contesti del tutto esterni al tema della staticità» ([1997] 2001, 384).

It is not evident whether the cover of Saint-Etienne derives from autonomous French research into the extreme development of the gothic culture or can in some way be connected to the English experiences, as hinted by Viollet-le-Duc ([1854–1868] 1997, 4: 122). In my opinion, the Anglo-Saxon domination in Normandy during the Hundred Years War (1417–1450) does not explain the migration of this constructive technique, not very diffused even in England. In fact, it is very difficult to imagine the English deliberately exporting this type of vaults, above all in light of the policy inaugurated by Henry V (treaty of Troyes of 1420) and continued by the duke of Bedford of presenting the king of England not as a conqueror but as a legitimate descendant of the French king Charles VI. Therefore there was no political advantage in imposing an English constructive tradition in France.

THE EXPANSION OF THE CONSTRUCTIVE SYSTEM OF THE *VOÛTES PLATES DALLÉES* IN NORMANDY (1516–1552)

The covers of the lower chapel of Gaillon and of the porch of Saint-Etienne utilize the same static system, even if they reach different formal results. From these two structures, classifiable respectively as «full arcs-diaphragm» and «fret-worked arcs-diaphragm», originates the greater part of the other *voûte-plates dallées* covers that were built in Normandy until 1560.

Full arcs-diaphragm

In the chapel of the bishop's palace in Bayeux, commissioned by the Veronese Ludovico di Canossa between 1516 and 1531, the system of Gaillon is applied to an octagonal plan. The chapel is directly accessible from the new gallery which starts from the bishop's apartment, in conformity with a model that will become current in the distribution of the French dwellings of the first half of the 16th century (CAF 1908, 1: 174; Chatenet, 2001). The full *arcs-diaphragm* are disposed in four couple of parallel arches that join the vertex of the octagon, Figures 8.

The frescoes have been added in the 17th century. The diagonally-traced ribs indicate that this building still belongs to the late gothic tradition. But, as pointed out by Chatenet (2001, 388), the fact that this type of structure is used in one of the first Renaissance castles of France, Gaillon, and in the residence of a sophisticated Italian humanist such as Ludovico di Canossa, determines the immediate success of the model, which is at once considered a Renaissance innovation.

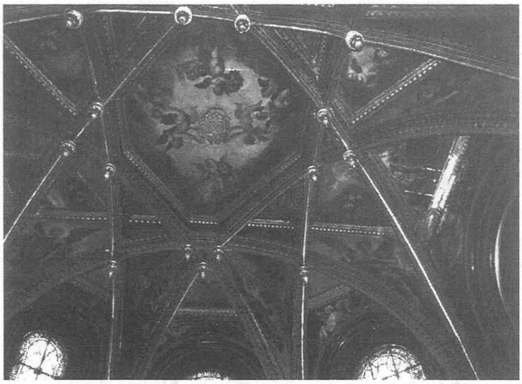

Figure 8
Chapel of the bishop's palace in Bayeux (1516–1531). View of the interior with the four couples of *arcs-diaphragm*

Several full *arcs-diaphragm* supports the stone coffered ceilings of the Virgin's chapel of Sainte-Hilaire in Tillières-sur-Avre, realized in 1543. Figures 9

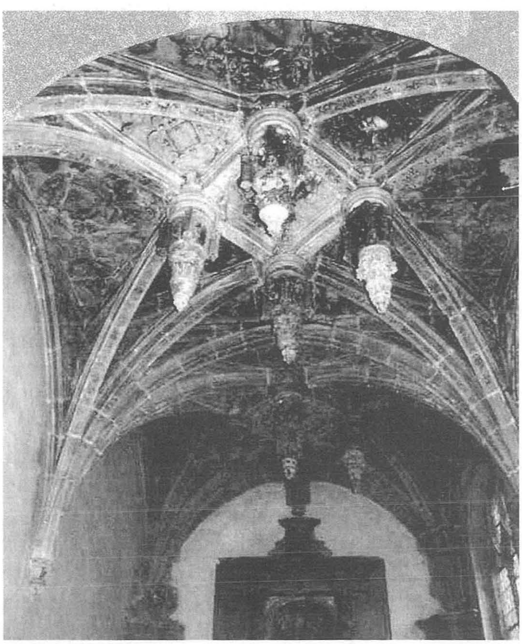

Figure 9
Church of Sainte-Hilaire in Tillières-sur-Avre. View of the interior of the Virgin's chapel (1543)

The coat of arms of the cardinal Jean the Veneur are represented in the chapel, therefore he could be the patron of this part of the church (Mouton, 1926, 23). In the chorus, datable to 1546, there are some fret-worked *arcs-diaphragm*. The structure-bearing ribs and formerets of both rooms still follow the gothic constructive logic, but the decoration is by now totally Renaissance: the gothic drawing of the ribs has been transformed in buttress of pier decorated with candelabre; the stone flags of the ceiling and the classic bas-reliefs adorning the parting walls employ forms derived from the repertoire of classical antiquity; in the chorus the fret-working is obtained using a small series of full-centre arches, built over some little Corinthian square ashlar piers.

A series of large full *arcs-diaphragm*, disposed on parallel, characterizes the cover of the porch of Notre-Dame at Vétheuil, in the Normand Vexin. Figures 10.

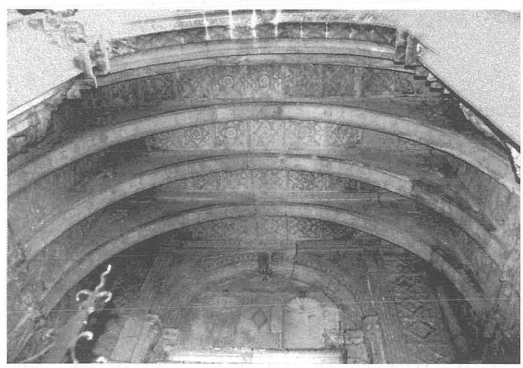

Figure 10
Church of Notre-Dame of Vétheuil. Porch (1540–47). View towards the vault with the parallel *arcs-diaphragm*

The porch, showing the coat of arms of Louis de Silly and Anne de Montmorency-Laval, is dated by Pérouse de Montclos to 1540–47 (Regnier 1909–1910; Pérouse de Montclos 1992, 715–717). The segmented arches are built over pilasters, which articulate the two sidewalls alternating themselves with full-centre arches niches. The stone flags of the ceiling are decorated with regulars forms that imitate coffers and ceiling reeds.

Fret-worked arcs-diaphragm

The prototype of Saint-Etienne-le-Vieux is first repeated in the radial chapels of the ambulatory of Saint-Pierre in Caen, constructed between 1518 and 1545 (CAF 1908, 73–75). Here, the theme of the fret-worked arc-diaphragm, is developed exalting the structural and decorative aspects. Every chapel has a different structure of liernes organization, which spaces from the rib-groined vault to the tracery vaults with *retombé central* or with a crown rib. The bearing ribs, conceived as «archi volanti», are always connected by secondary decorated ribs. Over the bearing structure the carved stone flags fill up the spaces between the complex game of the branches. All the structure is really articulate, and it is very difficult to recognize the role of the bearing arches, leading to the structural ambiguity so strongly disapproved by John Ruskin three centuries later .

An analogous situation is found in three of the ambulatory chapels of Saint-Jacques in Dieppe, built between 1525 and 1543 (Legris, 1918; CAF 1926, 251–279; Cahingt, 1983). Originally, also the Virgin's chapel, situated on the longitudinal axis of the chorus and destroyed by the English bombardments of 1694,[5] had a *voûtes-plates dallées* cover, as we can see in the description of David Asseline of 1682 (italics added):

> Voûte faite de pierre, très délicatement façonnée, *aussi plate qu'un plancher*; retenant neanmoins six culs de lampe très gros et très longs, chacun desquels est chargé de quatre images de hauteur d'homme (Legris, 1918, 96).

The vaults of the Virgin's chapel have been reconstructed in the 18th century with normal rib-groined vaults and, currently, the *voûtes-plates dallées* cover only the chapels of Saint-Nicolas (ex Saint-Michel), Figures 11, Saint-Yves (ex Saint-Jérôme) and Notre-Dame des Sept Douleurs. They were all constructed between 1525 and 1550, commissioned respectively by the shipowning families Guilbert, Ango and Saint-Maurice. In this instance the attention of the constructors focuses on the complex multiplication of the ribbings and not on the decoration of the stone flags.

A cover of the same type characterizes the Virgin's chapel of the abbey of Valmont, commissioned by Jean Ribault, Abbot from 1517 to 1552 (CAF 1926,

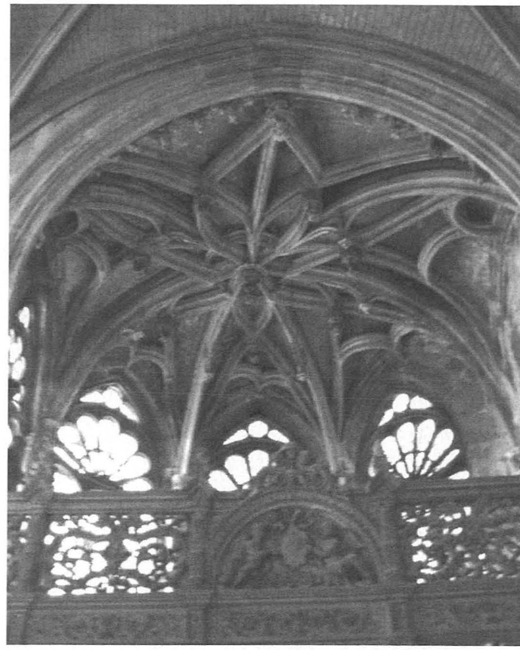

Figure 11
Church of Saint-Jacques in Dieppe. Saint-Nicolas Chapel (1525–1550). View towards the vault, with an extensive use of arcs-diaphragm

387–404; Mere 1979, 7–8). Human figures, foliage, shells and scroll ornaments decorate the ceiling of the chapel.

An opposite result, focused on the research of the stone coffered ceiling, is found in the splendid covers of the radial chapels of Notre-Dame-des-Marais at La Ferté Bernard, in the department of the Sarthe, Figure 12.

In this case the geographic location appears eccentric, as the Sarthe is outside Normandy, but the presence of the *voûtes plates dallées* in this zone is easily explained by taking into account the larger area occupied by the region in ancient times.[6] The construction of the three chapels, by the *maîtres-maçons* Jérôme Gouin and Jean Texier, began around 1524 but the covers have been realized just in 1543–44 by other hands, since Gouin and Texier died respectively in 1526 and 1531. The structure and the mature Renaissance decoration are due, most probably, to Mathurin Delaborde, in charge of the

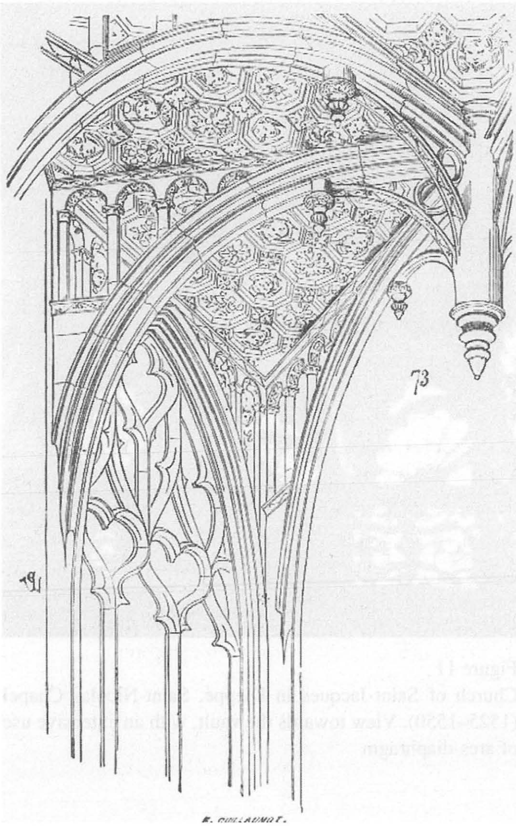

Figure 12
Church of Notre-Dame-des-Marais in La Ferté Bernard. Perspective view of the Sacré Coeur Chapel (Viollet-le-Duc, [1854–1868] 1997, 4: 123)

building yard from 1535 (CAF 1961, 225). Both the axis chapel, dedicated to the Très Saint Sacrement, and the two lateral ones, dedicated to the Sacré Coeur on the left and to Saint-Joseph on the right, open on the ambulatory with a clean span of 6,35 meters. The pointed arched with crown ribbing and *retombé central* and the fret-worked *arcs-diaphragm* are conceived with a clearer logic of the structure than the examples of Caen and Dieppe.

In fact, the juncture between the stone ceiling and the ribs is assured by a series of small arches built over column or by some architraved columns, very similar to those of Saint-Hilaire in Tillières. These columns distribute uniformly the weight of the ceiling on the bearing ribbings. From the main arches to the crown rib other liernes complicate the structural game. Exquisite and varied carvings of the stone ceilings add to the technical-formal complexity of the ribbings. In this building a perfect imitation of the wood coffered ceilings is achieved. In the chapel of the Sacré-Coeur the frame designs octagons and small rumbles, decorated with vegetable forms and human faces. In the first two spans of the axis chapel Greek crosses and squares form a regular sculpted texture, while in the apsidal part the carved coffers assume a round shape. In the Saint-Joseph chapel, finally, where the geometric design of the ceiling still follows the trapezoidal plan determined by the bearing ribbings, the game of regular geometric shapes is obtained by the chromatic alternation of black and white, Figures 13.

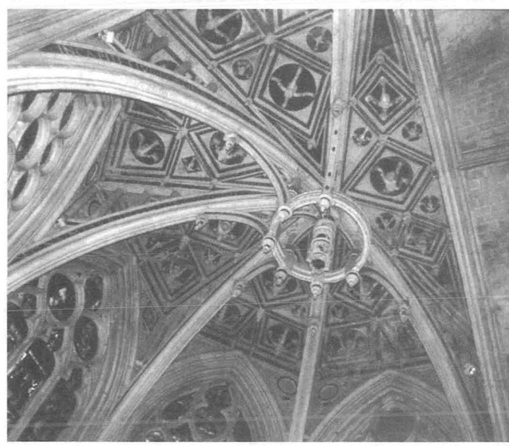

Figure 13
Church of Notre-Dame-des-Marais in La Ferté Bernard. View towards the vault of the Saint-Joseph chapel, with the chromatic decoration of the ceiling

The Bouton chapel in Beaune

The examples examined up to now follow the prototypes of Gaillon and Saint-Etienne-le-Vieux, without introducing substantial modifications to the static conception of the structures. A remarkable step

forward is done in the realization of the chapel commissioned by the canon Jean Baptiste Bouton in the collegiate church of Notre-Dame in Beaune, in Burgundy, built between 1529 and 1533 by the *maçons* Jean and François Lejay (CAF1928, 290–302). The cover of the chapel perfectly reproduces a stone coffered ceiling, without showing any elements of support. It seems to be resting only on the Renaissance consoles that run all over the perimeter of the room, Figures 14. But the static characteristics of the stone and the dimensions of chapel (600 × 495 cm.) naturally leads to exclude such solution and indicates *(suggests)* instead a system of flat arch straight vaults (Pérouse de Montclos, 2001, 163).

But the stone ceiling of the Bouton chapel is not a straight vault. Here the constructive technique of the *voûtes-plates dallées* cover is used in order to realize a perfect imitation of the wood coffered ceilings, the formal model of which is traceable, most probably, to the coffered ceiling of the palace of Justice in Dijon, commissioned by the king Francis I in 1522.[7] This ceiling covers a 17 × 12 meters room and is composed of 35 regular coffers, decorated with ceiling reeds. We can distinguish it from contemporary French examples for the absence of gothic ribbings and for the full Renaissance decoration.

The more interesting aspect of this cover, however, is not the perfect imitation of the wood ceilings, but its structural conception: the idea of using the rib-groined vault as a support for the ceiling of the *voûtes plates dallées*, is reinterpreted by transferring the bearing structure beyond the extrados of the cover. Above the ceiling, the branches of a rib-groined vault, completed by tie inverted arches, are completely hidden from the observer's view. The stone flags are suspended from a series of tie beams. The anchor bolts are concealed in the Renaissance ceiling reeds. The outward appearance of the building is adapted to its the interior structure: a fine carved gallery hides the view of the complex structure, Figures 15.

In practice, the system of the *voûtes-plates dallées* cover elaborated in Normandy has been freed from its gothic elements in order to realize an innovative, totally Renaissance, stone coffered ceiling.

Figure 14
The Bouton chapel in the church of Notre-Dame in Beaune (1529–1533). View of the interior towards the stone coffered ceiling

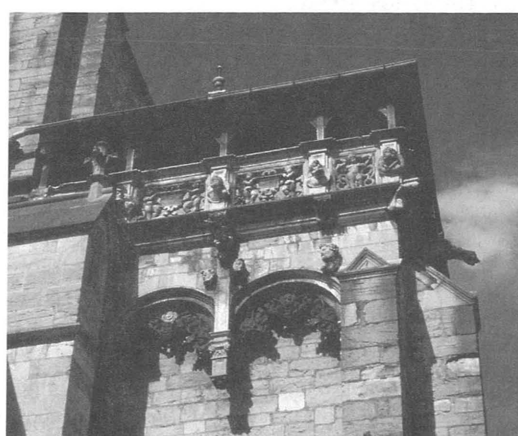

Figure 15
Bouton chapel in the church of Notre-Dame in Beaune (1529–1533). View of the exterior of the chapel, with the sculpted gallery hiding the structure of the vault

The transfer of the Norman system to Burgundy is due, most likely, to the patron, who was familiar with Norman architecture, in particular with the buildings of the d'Amboise family. In fact in 1512, Jean Baptiste Bouton, who already held several appointments in the Burgundy diocese, became in 1524 general vicar of Georges II d' Amboise,[8] archbishop of Rouen. In this capacity he certainly visited the chapel of Gaillon and, most probably, the other Norman buildings which employed the *voûtes-plates dallées* cover.

In my opinion, the development of a full Renaissance style can be attributed, at least in part, to the particular cultural atmosphere that prevailed in Burgundy, in particular the region of the Côte d' Or, in the 1530's. In fact, from 1529 is present in Dijon Claude de Longvy de Givry, bishop of Langres, descendant from one of the noblest families of the region, named cardinal in 1533, fine diplomat and deep connoisseur of Italian art. As early as 1510 his artistic and literary interests are pointed out in the dedicatory epistle of the *Metamorphoses* of Pierre de la Vigne.[9] Similarly in 1520 Jean Fustaillier dedicates to Givry his *De urbe et antiquitatibus Matisconensibus*. Moreover there is a possibility that the artists, who worked in the Bouton chapel, are the same who made the new Renaissance decoration of the chapel of the castle of Pagny, commissioned by the cardinal between 1535 and 1538 (CAF 1929, 305–316; David, 1929).

Conclusions

The fortune of the *voûtes-plates dallées* in France and in Normandy ended around the middle of the 16th century, excluding the isolated case of the cover of the aisles and the ambulatory of Saint-Germain d'Argentan, realized between 1600 and 1610 by Jacques Gabriel, who also worked in Saint-Pierre in Caen in 1603. The development of stone-cutting techniques and the experimentations of Philibert Delorme led French architecture to other accomplishments. In similar fashion the solution adopted in the Bouton chapel does not seem to have been followed neither in the region neither elsewhere in France. In both cases these structures are hybrids developed in that delicate period of transition between *flamboyant* and the Renaissance and characterized by formal inventiveness and structural audacity. They also testify the will and the ability of the French craftsmen to absorb and adapt new stylistic features to their technical base: they imitated the transalpine shapes but also invented new solutions, integrating two different static systems and finally arriving to the spectacular artifice of the Bouton chapel: «en s'affranchissant de la routine dans laquelle se tenaient les maîtres du XVe siècle, ils appliquèrent aux formes nouvelles les ressources de l'art de la construction du moyen âge» (Viollet-le-Duc, [1854–1868] 1997, 4: 124).

Notes

1. The first had been seen by the French army in palazzo Venezia in Rome during the campaign of 1494–1495 and the second was introduced by Riccardo da Carpi in the castle of Gaillon in the first years of the 16th century.
2. For example, the vaults of the Saint-Thomas' chapel in the collegiate church of Nantes, of the collegiate church of Saint-Jean at Montrésor, of the churches of Sepmes and Villiers-au-Bouin. The eastern gallery of the cloister of Saint-Martin in Tours is covered by a series of hanging domes, where the ribs form the regular compartment.
3. For example, the vaults of the court's gallery of the castle of La Rochefoucauld but also the covers of the chapels Poitiers, Cytois and Fresneau in Notre-Dame la Grande in Poitiers.
4. I am grateful to Pascal Leroux, who guided me among the ruins of the church, sharing with me his unpublished studies on the building. Thanks to Yves Lescroart I have been able to visit the parts of castle of Gaillon normally closed to the public..
5. The night of July 22th 1694, Lord Berkeley, commander of the British fleet, ordered the bombardment of the city.
6. The carved hanging keys, for example, are derived directly from those in the palace of Justice of Rouen, thus confirming the influence of Norman architecture in the neighbouring regions.
7. The king of France held in Dijon a «lit de justice» in 1521 and would have donated to the city the ceiling the following year (Fetu, 1872, 43–45; Caf 1928, 305).
8. Georges II d'Amboise became archbishop of Rouen three years after the death of his uncle Georges Ist d' Amboise (1510), the client of the castle of Gaillon.
9. *P. Ovidii Nasonis Metamorphoseos libri moralizati : cum pulcherrimis fabularum principalium figuris Ovidii quindecim Metamorphoseos libri . . . par reverendum*

patrem magistrum Petrum Lavinium philosophum poetam ac theologum . . . Anno Domini millesimo quingentesimo decimo XV kal. maii.

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Analysis and repair of historical roof structures: Two examples, two different concepts

Rainer Barthel
H. Maus

Two historical roof structures are presented. The history and sequence of alterations, damages and deformations during life time is described. The history is the key for understanding the damages and for developing the appropriate repair concepts. The two structures are similar but the alterations they suffered are different. A suspension truss is the main feature of both structures. Both structures show big deformations, but caused by different reasons. Two different concepts of repair measures are derived. The advantages and disadvantages of the measures are discussed.

Introduction

Historical roof structures are an important part of the cultural heritage. The objective of every repair concept should be to secure the statical stability, to minimise the alterations of the historical substance and to preserve the architectural form. The basic requirement for the development of a repair concept is the precise understanding of the damages and deformations.

Damages can have a lot of different reasons. A precise and detailed assessment of the damages is essential. In a lot of cases the history and the sequence of alterations and deformations during the life time has to be reconstructed in order to get an explanation for the present situation. Statical calculations considering different stages of alterations can help to understand the sequence of reactions.

Roof structure of a town house in Munich

A model of Munich dated 1572 shows the building of Burgstrasse n° 8. It is the building with the huge roof (Fig. 1). The roof structure still existing today was erected in 1615 and has the same shape as the roof in

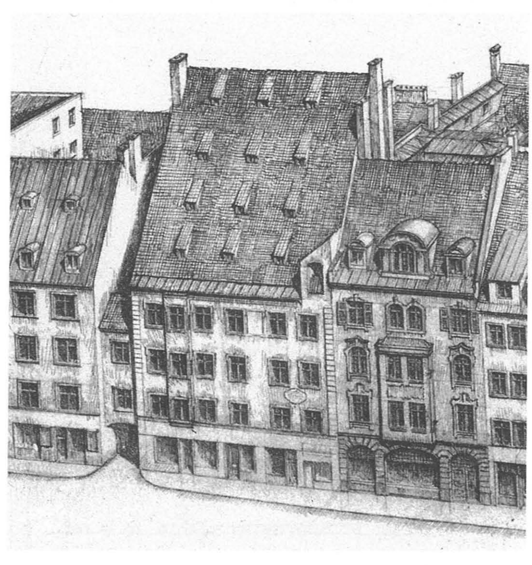

Figure 1
Townhouse Burgstrasse 8 in Munich. Drawing based on a town model of Sandtner dated 1572 (Häuserbuch der Stadt München)

the model. It is completely original. The structure is shown in Figure 2 and 3. There are four storeys inside the roof. The struts at the first floor are not part of the original structure. They were added in the last century. The free span of the was 21,10 meters from the eastern to the western wall of the building. The distance between the north and the south gable wall is 14,30 meters. The height of the roof structure from the eaves to the ridge is 13,90 meters.

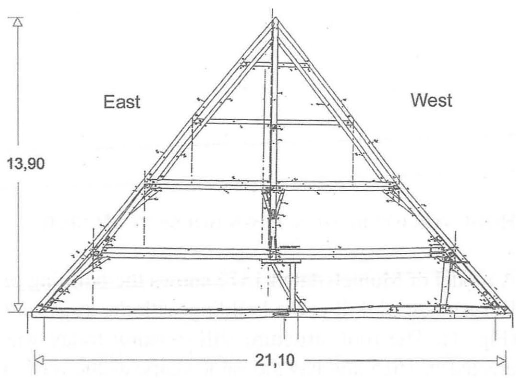

Figure 2
Cross section (Drawing of Franz Hölzl, Munich)

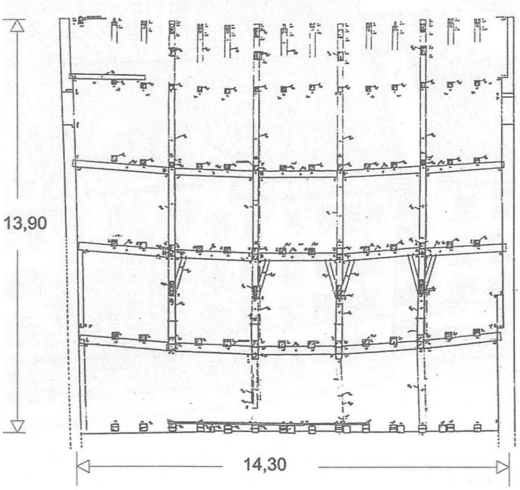

Figure 3
Longitudinal section (Drawing of Franz Hölzl, Munich)

The structure consists of 14 rafters at each side connected by horizontal beams at five levels. In addition four principal trusses stiffen the structure. Above the first storey a continuous beam with a cross section of 27 by 30 cm is spanning in longitudinal direction between the gable walls. This beam is hanging at king posts which are connected to a suspension truss in the third and fourth storey (Fig. 4). In the year 1726 an additional truss was built in at the first storey parallel to the west side (Fig. 5).

The damage assessement shows the following:

1. Along the eaves on the west all joints between the ends of the rafters and the anchor beams are rotten.

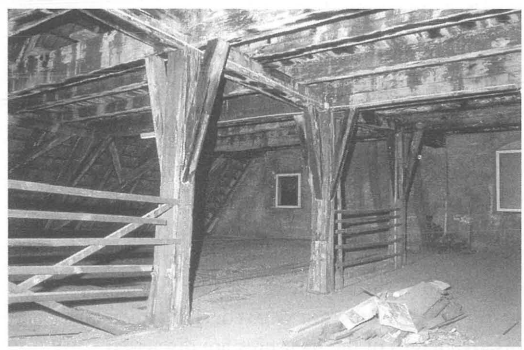

Figure 4
Second storey inside the roof structure, the king posts in the middle axis

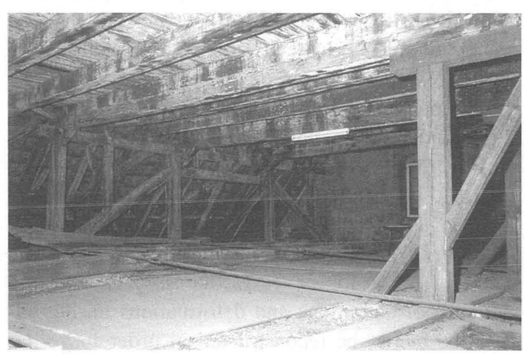

Figure 5
First storey inside the roof structure, the truss visible in the background dates from 1726

2. A lot of joints are drawn apart, especially in the upper part of the structure. Wooden nails are broken.
3. There are big deformations of the upper floors. The sag between the gable walls is 42 cm (Figs. 3 and 4).

Analysing the damages and the deformations joint by joint it was possible to reconstruct the history of the roof since the erection.

Soon after the erection damages along the eaves at the west side occurred due to a leaking roofing (Fig. 6a and Fig. 8). The joints between the rafters and the anchor beams were destroyed and failed. The rafters slided outwards and down to the masonry. The entire roof settled and got into an inclined position.

About a hundred years after the erection of the roof the truss was added in order to stop the movements. This truss took the big vertical loads of the roof and prevented the roof from further vertical movements. However the truss caused additional damages. The truss is a vertical support not a horizontal one. The horizontal thrust of the roof had no support. A new mechanism was created (Fig. 6b). The entire roof moved to the west. The truss got inclined. At the east side the movement to the west also caused a vertical movement. The lower support is a centre of rotation for the struts of the principal trusses. This movement was hindered only by the rafters. The rafters are continuous beams going from the eaves to the ridge. As a result the connection between the rafters and the principal trusses were drawn apart (Fig. 9). The wooden nails failed.

At the end the settlement of the upper floors was 42 cm. The struts were built in order to create a support in the middle of the span (Fig. 6c). The self load of the roof is now supported by these struts. The forces in the suspension structure changed from tension to compression. Some joints which were designed to take tension forces only could not take compression forces and fell apart.

The deformations described occurred in the middle between the gable walls. At the gable walls the movements are restricted because of the connection of the structure to the masonry. A complicated three dimensional deformation figure was created (Fig. 7): At the west side all rafters slided downwards. Braces connecting the gable wall and the structure failed. At the east side there was no settlement at the eaves. The horizontal beam at the second level, running from gable wall to gable wall, was bent but did not fail. The deflection line shows a horizontal and a vertical component. In a similar way the heavy beam in the middle was bent. It is also fixed at the gable walls. Therefore the three dimensional deformation figure

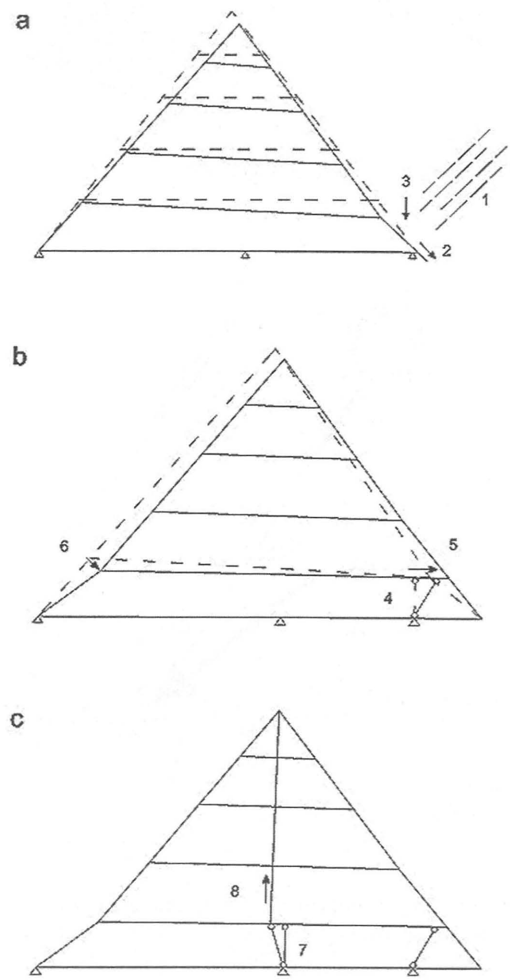

Figure 6
Sequence of damages and repair measures 1) leaking roofing 2) destroyed connections between rafters and anchor beams 3) vertical deformation 4) new truss in order to stop the deformation 5) horizontal deformation 6) rotation of the inclined strut 7) new columns in order to stop the vertical deformation 8) compression instead of tension

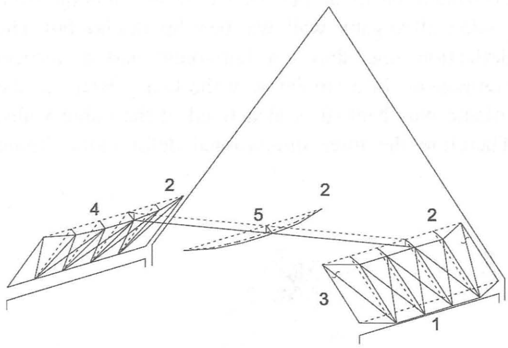

Figure 7
Three dimensional deformation figure

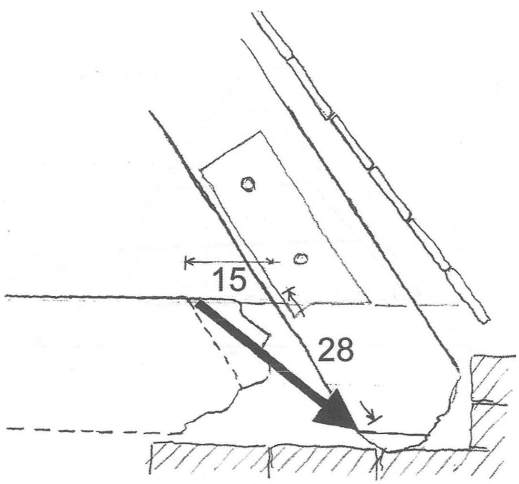

Figure 8
Damages at the west side

shows a big settlement only in the middle. Deflection lines due to bending are visible only in the longitudinal section (Fig. 3) but not in the transverse section (Fig. 2). The structure was prevented from collapse mainly by the horizontal beam at the middle axis.

On the basis of these results the repair concept, which was proposed, intends to push the structure back to the original position and to take out the big deformations. Experience shows that it is practicable. However it is expensive. It is not yet realized. It will make it possible to restore the original statical performance without introducing a lot of additional elements. In the present situation it would be nearly impossible to repair all the joints which are drawn apart. Big steel elements would have to bridge the gaps. Removing the structure into the original position it will be possible to bring the joints together again. Then it will be possible to repair the joints in a reasonable way. The forces will be taken out of the truss along the west side. However it should remain in the roof. The struts in the middle will be taken out. A statical analysis of the original structure shows that it will be stable. Only a few additional elements are necessary in order to secure a sufficient safety factor under wind and snow loads.

ROOF STRUCTURE OF THE «ALTE HOF» IN MUNICH

The historical roof structure of the «Alte Hof» in Munich is the second example (Fig. 10). The main feature of the structure is a suspension truss, too. The «Alte Hof» is a castle founded in the 12th century. It has been the residence of the German Emperor Ludwig the Bavarian from 1328–47(Burmeister 1999). The oldest part of the historical roof dates from 1525 and is one of the oldest original roof structures in Munich. The part which is presented in this paper dates from 1562 (Figs. 11 and 12).

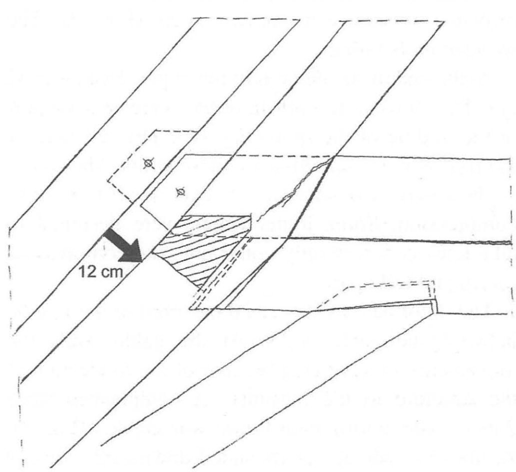

Figure 9
Joints drawn apart at the east side

Figure 10
«Alter Hof» Munich

In the original situation the roof structure had a free span of 13,35 meters. It had to take the load of the ceiling above a huge hall. The ceiling is rather heavy due to a filling between the beams. The beams are supported in the middle axis by one big upstand beam. Its length is 18m and its cross section is 40 by 30 cm. This beam is hanging at suspenders consisting of wrought iron. They are anchored at the suspension truss situated in the upper storeys.

There are three storeys inside the roof structure. In the first storey the struts of the principal trusses are inclined parallel to the rafters. The suspenders which are made of wrought iron instead of timber and the ornaments at the braces are indicating that the first storey was considered as a space of high value. The construction of the ceiling is also rather heavy and was painted.

In the second storey the struts of the principal truss are in a vertical position. Braces are connecting the horizontal beams and the rafters. In the middle axis there are the king posts and between them a very stiff bracing in longitudinal direction. A lot of elements are decorated by wood carving. Even the joints themselves are carved work. In the third storey suspension trusses take the vertical loads of the middle axis.

At a later time a wall was erected in the storey underneath the roof exactly in the middle of the structure. The new wall acts as a support for the ceiling and the first storey of the roof. It takes out the forces of the suspenders.

The damage assessment of the present situation shows the following:

1. local damages due to a leaking roofing, mainly at the west side
2. big deflections at all the floors
3. broken joints due to overloading, especially at the connection of the braces and the rafters in the second floor (Figs. 13, 14).
4. local damages caused by fire. These damages are dated from the time of second world war. The repair was done very roughly.

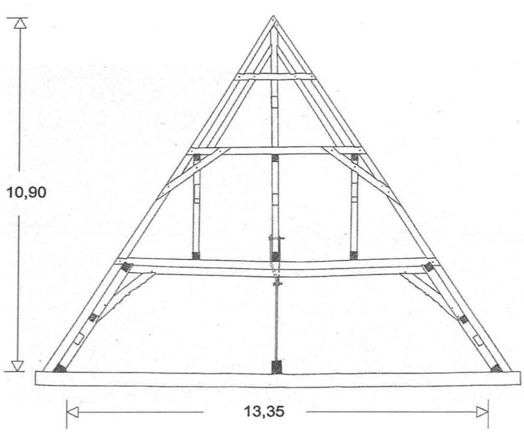

Figure 11
Cross section of the roof structure

Figure 12
First storey inside the roof structure

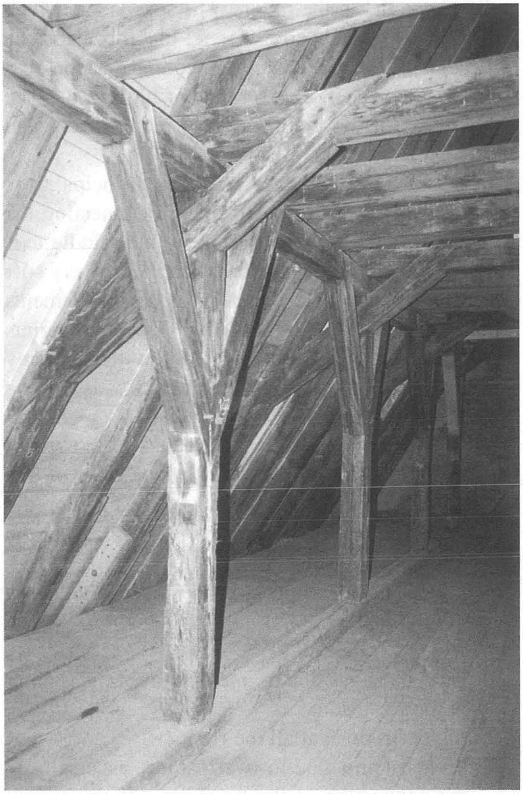

Figure 13
Second storey inside the roof structure, connection between rafter, brace and vertical strut

The analysis of the statical behaviour and the seqence of alterations and damages lead to the following conclusions (Fig. 15):

During the first period of time the structure had to span 13,35 meters and had to take the entire load of all floors. The suspenders and the suspension truss itself were strong enough. But there were problems in transferring the forces to the outer supports because of the distribution of stiffness inside the overall structure. The load path with the highest stiffness is the path from the suspension truss directly to the braces and to the rafters. The principle truss is not as stiff as the rafters due to the deflections of the horizontal beams which support the vertical struts. However the joints between the braces and the rafters were not strong enough. They are halved joints with carving. Deformations occurred and the principle truss had to take over the loads. Bending moments caused big deflections. A statical calculation, considering the damaged joints at the rafters, confirms that.

Probably the deflections were the reason for the wall which was built underneath. The wall took over the loads of the floor. The forces at the suspenders and the suspension truss were reduced. However further deformations occurred and the suspenders buckled.

At the beginning of our work it was intended to strengthen the suspension structure in order to establish the original flow of forces. The consequence of that would have been to replace the destroyed joints or to introduce an additional steel frame. To replace the joints was unacceptable because of the carving works. Even a copy of the original joint would not be sufficient. The acting forces at this point require a total different type of joint.

The result of a lot of discussions was extremely simple. Upon the upstand beam in the first storey along the middle axis a few small steel columns were positioned. They take the loads of the upper floors and transfer them directly to the middle wall underneath. The king post is now under compression instead of tension. The compression is very small and the design of the joints make it possible in this case. The stiffness of the entire roof structure became much higher even against horizontal loads. No further

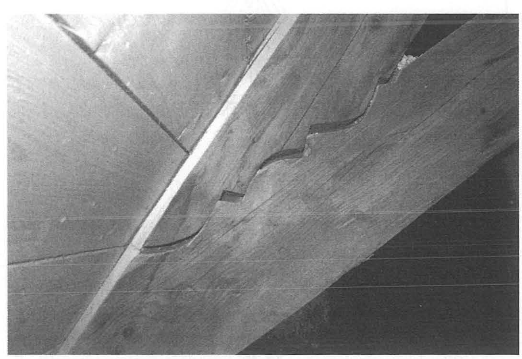

Figure 14
Overloaded joint between rafter and brace in the second storey

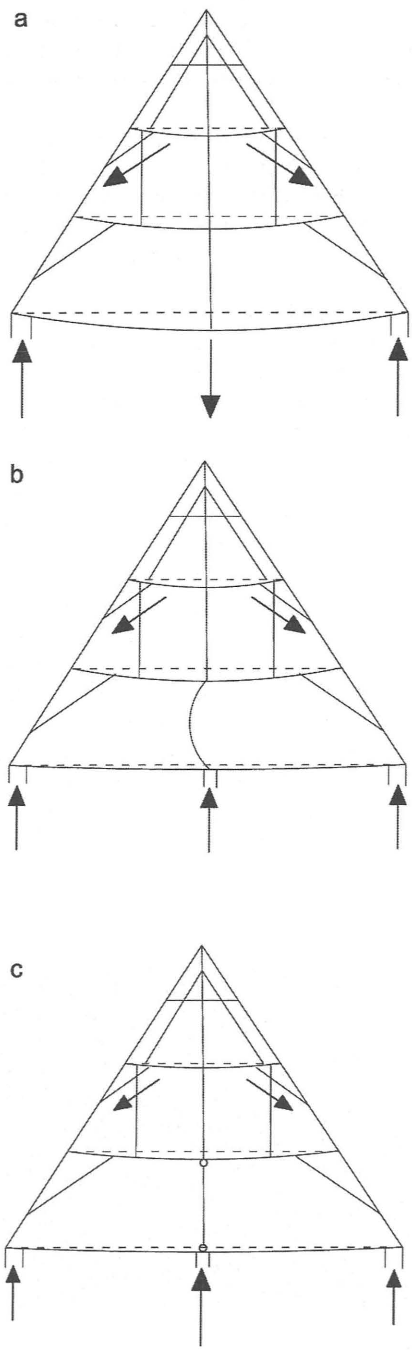

Figure 15
Sequence of statical systems

measures were necessary. The overloaded joints could stay in place without repair. They are a part of history.

The disadvantages of this solution are:

— The original flow of forces is altered. From a engineering point of view it could be desirable to re-establish the free span. That is a historical value as well. On the other hand it has to be stated, that the structure never really worked like that. Originally it was intended but the joints were too weak.
— The columns disturb the big space at the first storey. From an architectural point of view this is not acceptable, even if the space is not used anymore. In this case, the space was already divided into two parts in former times. A few timber elements are still in the middle of the room and they remain there. The columns are not the only elements which disturb the space.

Conclusion

The examples demonstrate the importance of a precise assessment of the damages. The history of the alterations and of the damages can be the key for the understanding of the structure. The statical analysis has to consider different situations. The objective of repairing old structures is not necessarily to re-establish the original situation. The repair measures are a further step in the history of the structure.

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Static analysis and evaluation of the construction system of a gothic «choir-window» consisting of a filigree tracery and slender stone rips

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L. Schiemann
M. Jagfeld

The St. Georg church at Nördlingen located in the south of Germany shows a large gothic tracery window with a hight of more than 12 metres. Damages at the stone bars and the tracery are the reason for an assessment of the structure and a statical analysis. The framework consists of sandstone and wrought-iron saddle bars. The stone bars are extremely slender. The analysis is carried out by means of the finite element method, considering the opening of the joints under wind load. A theoretical model of the principle statical behaviour is developed taking into account the sequence of construction and the stiffness of the structural elements.

Introduction

St. Georg at Nördlingen, located near Augsburg in the south of Germany, is one of the last churches of the late gothic period. Its significance is not only due to its great dimensions in height and length but also to its accurate technical construction. The height of the nave is 18.6m, the length is 93.5 m, the width 23.5 m. The pillars are extremely slender. The precise stonework of the large filigree gothic tracery also emphasizes the high quality of the building (fig. 1). The construction of the building was begun in 1427 at the choir and was completed already after 92 years in 1519.

Figure 1
St. Georg at Nördlingen with the tracery window in the middle

The Choir Window

Geometry and materials

The height of the window is nearly 12.30 metres , the width is 3.70 metres (fig. 2). At a level of approximately 12.0 metres a typical stump tracery (fig. 2, pos. 6) is constructed.

The framework consists of five slender stone bars - two main bars (fig. 2 pos. 1) and three minor bars (fig. 2, pos. 2) – each of them 7.85 m long. The five stone bars devide the glass front of the window into six stripes. Bars consisting of wrought iron (fig. 2, pos. 3), span from one side of the window to the other, piercing the stone bars at their joints.

At about half of the height a horizontal beam interrupts the over all framework (fig. 2, pos. 4). It provides an additional stiffness in horizontal direction. The assessment of the connection between the horizontal beam at the reveal proves, that additional stiffening was considered as necessary during construction time (fig. 6). It was not planned in advance.

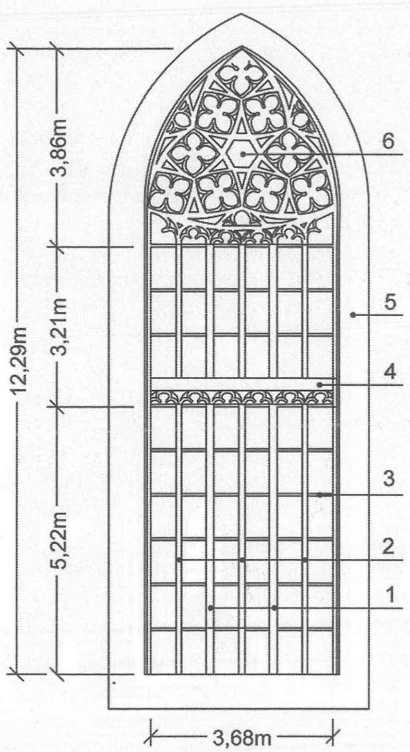

Figure 2
Choir window: 1 main bars 2 minor bars 3 saddle bars 4 central beam 5 reveal 6 tracery

The framework including the tracery consists of two different cross sections. The big one is about 15 cm wide and 30 cm deep, the small one is about 12 cm wide and 23 cm deep (fig. 3, 4). The horizontal beam consists of a tracery at the bottom and a full cross section at the top. It is 30 cm deep and 58 cm high including the tracery.

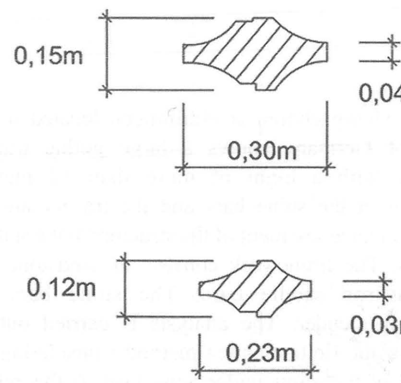

Figure 3
Cross-sections, top: main bar, bottom: minor bar

The material of the framework of the tracery and of the horizontal beam is a local sandstone.

In a course of tests at test pieces the following 'material constants' for the sandstone were determined:

- average compression strength: $\beta_m = 40.65$ N/mm^2
- average density: $\rho_m = 2.15$ g/cm^3

The wrought-iron saddle bars are conducted as rectangular cross sections (30x50mm) and have after the tests the following material indices:

- average tensile yield strength: $f_{y,k} = 247.2$ N/mm^2
- average ultimate strength: $f_{u,k} = 347.7$ N/mm^2

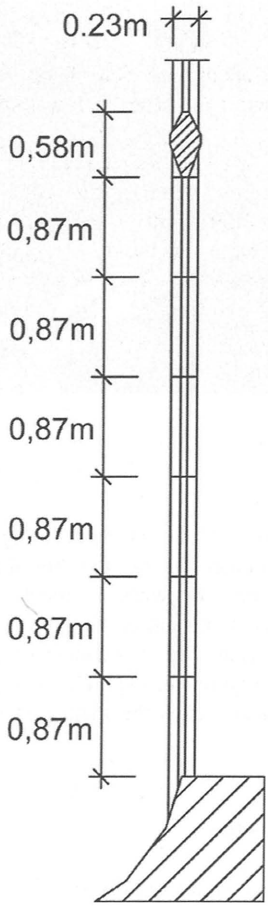

Figure 4
View of minor bar with joints (drawing true to scale)

The reveal (fig. 2, pos. 5) is made out of the very resistant local stone called suevit.

Figure 5 shows the structural joints and the segments of the tracery. The originally large sized individual segments of the tracery are a particular characteristic of the choir window.

Damages

The assessment of damages shows:

— some bars of the framework are not anymore precisely vertical,

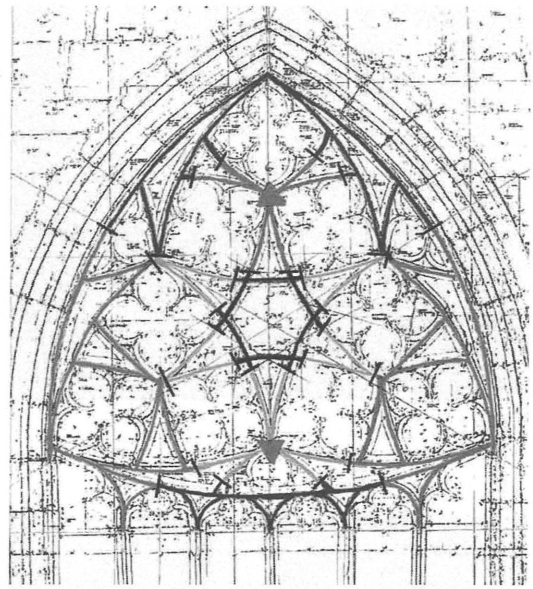

Figure 5
Cutting of stones of the tracery and structural joints

— at the connection between the horizontal beam and the reveal gaps and cracks occur (fig. 6).

— at the anchoring points of the saddle bars the stone is broken (fig. 7),

— originally joints of the tracery show gaps of about two centimetres width (fig. 8),

— some of the stones segments of the tracery are broken.

The main reason of the damages are horizontal movements of the lateral walls adjacent to the window. A deformation of the window arch or an enlargement of the width leads inevitably to disturbances of the structure of the tracery. Such movements can have different reasons. The horizontal thrust of a damaged roof structure or the thrust of the choir vaults, settlements of the foundation and vibrations due to an explosion in the second world war may have caused the movements.

STATICAL BEHAVIOUR

It is a very common assumption that the wrought-iron saddle bars take the wind load. That seems reasonable

Figure 6
Support of the horizontal central beam

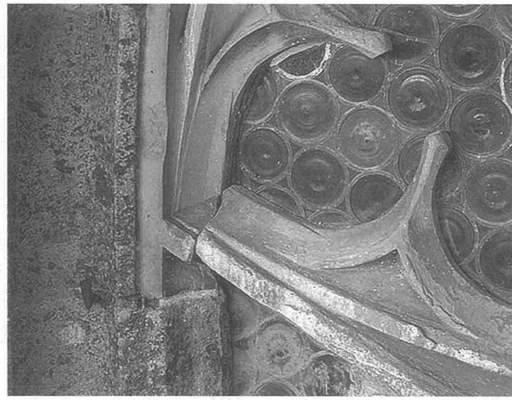

Figure 8
Broken joint at the tracery

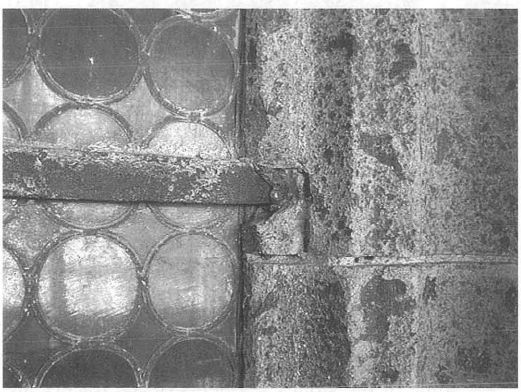

Figure 7
Anchoring of saddle bar

because of the extreme slenderness of the stone bars. The statical analysis of the saddle bars lead to a different conclusion.

Saddle bars

Assuming that the complete wind load is taken only by the saddle bars the stress due to the bending moment is 19 kN/cm². This value could be acceptable. However the deflection is very big. The maximum displacement is nearly 5.0 cm. It is approximately 1/73 of the span, far beyond 1/300 of the span, which is considered as reasonable for modern structures. However the deciding fact is, that such a displacement is far too big for the stone framework. The stonework is much stiffer. Big damages like crushing and spalling of the edges at the joints would occur. The assessment of damages do not show such severe damages. The conclusion is that the stone skeleton takes the main part of the wind load.

The stone skeleton

In the following, it is assumed, that the wind load is taken completely by the stone skeleton.

The statical analysis of the framework work is carried out using the finite element program ANSYS. The material model is linear elastic. The modulus of elasticity is assumed to 5000 kN/m². The opening of the joints is taken into account by an iterative method. The stiffness of the elements at the joints which show tensile stress are reduced to nearly zero. Then the calculation is repeated until the results show no significant tensile stress anymore. The analysis is based on the deflection theory. The iterative solution method takes into consideration the initial imperfections and big deformations. In order to determine a safety factor the wind load has to be increased up to the failure of the structure. It can occur due to material failure or due to snapping through. One bar is modelled using about 5000 solid elements. These elements are normally used for the

three-dimensional modelling of solid structures. They are defined by eight nodes having three degrees of freedom at each node (fig. 9). The simulation of the joints require the big amount of elements. The results, especially the reaction at the supports, are checked by comparative analysis methods as far as possible.

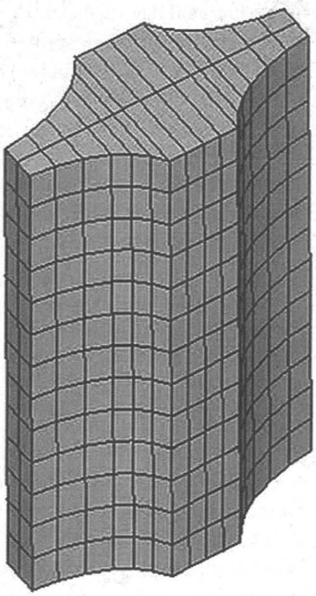

Figure 9
Modelling of a minor bar with three-dimensional elements

The overall structural framework can be subdivided into four separate substructures:

- the vertical bars below the horizontal beam
- the vertical bars above the horizontal beam
- the horizontal beam and
- the tracery at the top.

The slender bars of the framework require an axial compression force to be able to transfer wind load. Only in this case the bars can take bending moments. The axial forces can result from self weight and by creating a flat arch action between rigid supports. The rigid supports can be provided by the heavy masonry around the window. It has to be discussed later how to ensure the stiffness of these supports. In the following it is assumed that they are absolutely rigid.

Concerning the self weight it is necessary to distinguish two cases. If the tracery at the top is supported only by the vertical bars the maximum of the axial force is acting. If a significant part of the self weight of the tracery is supported by friction at the connection to the reveal the axial force is smaller. In this case the framework is stressed only by the self weight of the bars and of the horizontal beam (fig. 10). This is the worse case in view of the additional wind load.

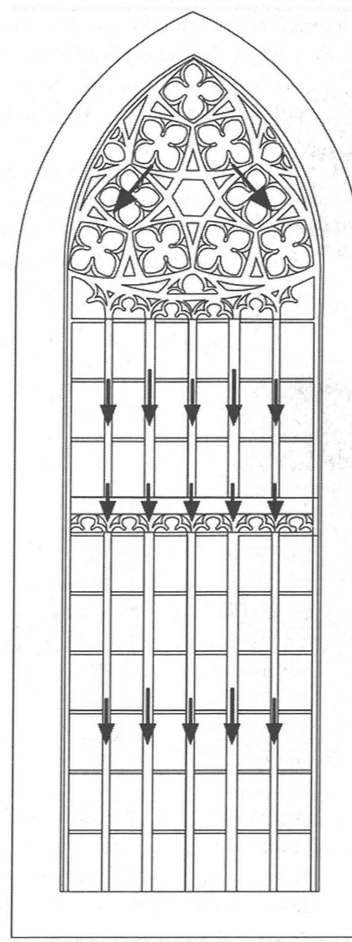

Figure 10
Load case minimal self weight

The analysis shows, that the self weight is not high enough to enable the vertical bars to take the full wind pressure or wind suction. A bar which is fixed at the bottom and supported only horizontally at the top is not stable under wind load. In addition to the axial forces due to self weight the forces due to the arch action have to be taken into account in order to get a state of equlibrium.

The structural model of the window is illustrated in fig. 11. It is assumed that the horizontal beam is acting as a rigid support for the bars in horizontal and vertical direction. The stiffness in horizontal direction is provided by creating a flat horizontal arch inside the beam. Therefore the masonry on both sides of the window has to be stiff enough.

The stiffness in vertical direction is provided by the heavy masonry upon and underneath the window. Regarding the vertical forces it is possible to assume a diaphragm action at the tracery. This assumption has to be proved separately.

The model works only on condition that all joints are very stiff and no gaps or cracks interrupt the flow of forces. The shear forces also have to be transmitted at the joints. Both preconditions are not unrealistic.

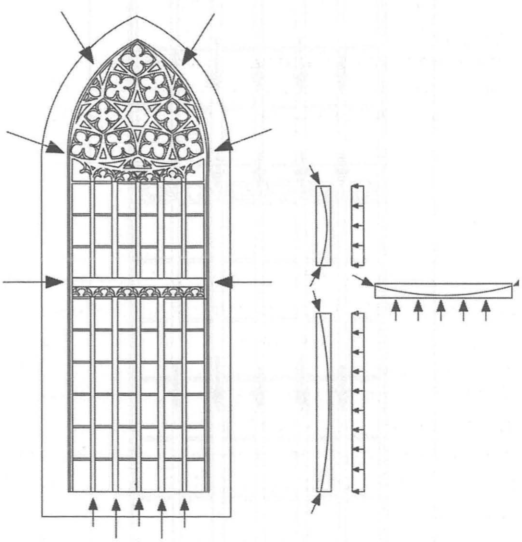

Figure 11
Structural model. Left: supporting reactions due to wind load. Right: vertical bars and horizontal beam subjected to wind load (principal diagram)

The assessment of the window shows that the joints between the stones are very thin even in cases their lead is used as fill. At the tracery there are dowels inside the joint to ensure a shear stiffness.

Statical analysis of a minor bar

In the following the analysis of the most slender bar below the horizontal beam is described and some results are given to serve as an example. Defining the statical system and its boundary conditions the sequence of consruction has to be taken into consideration. The framework of the window was built after the walls of the choir have been finished. Setting up the stones the bars get under compression and suffer an elastic deformation. Closing the last joint at the top of the window a situation is created which can be considered as a pre-stressed structure between rigid boundaries.

This situation has to be simulated in two steps using two different statical systems (fig. 12). In the first step the elastic shortening (u_z) due to self weight (G_N) and due to the weight of the elements above (G_1) has to be determined. Therefore the first system is fixed at the bottom and only horizontally fixed at the top (fig. 12a). The second system has rigid supports at the bottom and the top. In this system tensile stress is created at the top due to the self weight. Now the vertical displacement uz which was determined in the first step has to be imposed to the system. The result is a system which simulates the real situation between the rigid boundaries (fig. 12b). It is the starting point for subjecting the bar to wind load. The iterative analysis described above now has to be carried out.

The most important result is that a state of equilibrium under wind load is possible. The reaction forces in vertical direction due to self weight plus wind load are significantly higher than due to self weight only. The flat arch action is an essential feature of the statical behaviour. The joint in the middle open up to the half of the cross section. The stress distribution at the joints are given in figure 13. The maximum deflection is 0.5 cm.

In figure 12 and 13 the results are given for maximum self weight. In case of minimal self weight the reaction forces are smaller (G_N = 1.77 kN, G_1 = 2.88 kN, V_G = 4.65 kN, $V_{G+w,u}$ = 9.39 kN, $V_{G+w,o}$ = 5.84 kN). However the compressive stress is

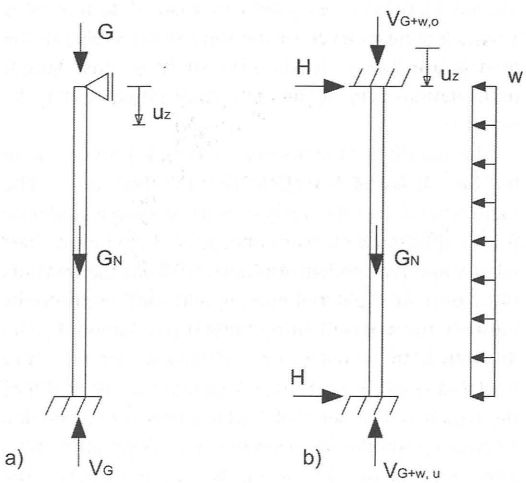

Figure 12
Resulting forces for
a) self weight $G_N = 1.77$ kN, $G_1 = 5.84$ kN, $V_G = 7.61$ kN,
b) self weight plus wind load $G_N = 1.77$ kN, $V_{G+w,u} = 10.25$ kN, $V_{G+w,o} = 6.71$ kN

greater ($\sigma_{max} = 10.0$ MN/m^2). The reason for greater stress under a smaller load is a deeper opening of the joints. This causes a great edge pressure.

The results depend very much on the value of the mudulus of elasticity. A great value leads to a stiff structure, small displacements and small compressiv stress due to a small excentricity of the thrust line.

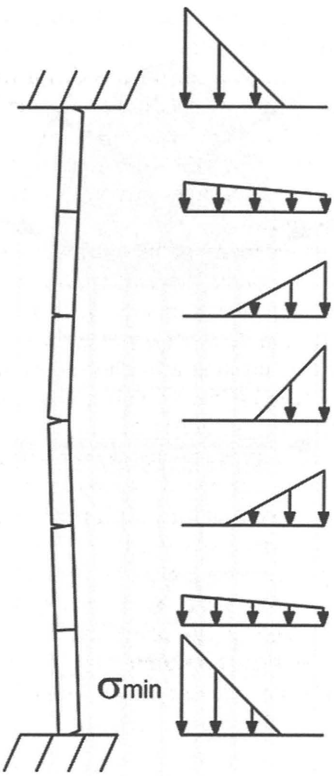

Figure 13
Compressive stress at the joints under self weight + wind load: Maximum compressive stress $\sigma_{min} = 7.9$ MN/m^2, maximum displacement in horizontal direction $u_h \sim 0.5$ cm

The horizontal beam

The horizontal beam acts as a horizontal support of the bars. It has to work like the bars. However there is a big difference. The beam is not pre-stressed by self weight. Only a flat arch inside the cross section and between rigid horizontal supports can take the load. The results of the analysis show rather small displacements and acceptable compression stress. However the resultant forces at the horizontal supports are about 30 kN.

Figure 14
Modelling of the horizontal beam (displacement 50-times magnified)

Conclusion

The theoretical model of a framework between rigid boundaries leads to reasonable results. Not only the straight bars but also the tracery at the top of the window will behave like that. At the tracery the flow of forces is more complicated. However it is very stiff in relation to the bars and allows different load paths. In figure 15 the most important ribs are marked. The hanging arch gives an additional stiffnes in vertical direction.

In reality the ironed saddle bars will participate to a certain extend. At the described window showing a width of 3.7 metres the saddle bars take only a little part of the wind load. Nevertheless the saddle bars are very important. At the first place they provide

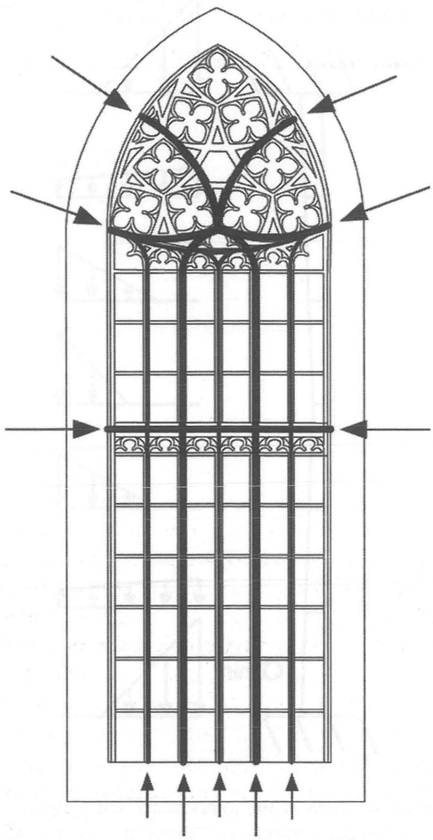

Figure 15
Load path and reaction forces

horizontal stiffness in two directions during construction. When the window is finished the saddle bars are essential for the stability of the bars in the plane of the window. In addition they provide a second load bearing system in case of failure of a stone element or even of the total stone skeleton due to extraordinary loads. In such a case great deformations will accur but a total collapse may be avoided.

The stability of the tracery window is proved due to the fact of its age of about five hundred years. The main objective of the analysis is to develop a guideline for an appropriate repair concept of the window. The most important conclusion derived from the analysis is to provide rigid boundaries and stiff connections between the stones. During the last five hundred years the overall movements of the masonry of the choir lead to a small but significant increase of the width of the window. The described arch action is reduced due to cracks and gaps between the stone especially at the horizontal beam. Repairing the framework of the window at the first place stiff connections between the stone elements have to be restored. In addition the masonry of the choir has to be assessed thoroughly. It has to be ensured that the masonry is not subjected to a horizontal thrust of the roof structure. The horizontal thrust of the vaults in relation to the stiffness of the masonry has to be examined. It has to be discussed whether a horizontal tie rod at the level of the horizontal beam should be inserted in order to pre-stress the beam or at least to take the forces.

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The «Torre Grossa» in San Gimignano: Experimental and numerical analysis

Gianni Bartoli
Paolo Spinelli

The «Torre Grossa» (literally *big tower*) is the most relevant within the 13 towers which, nowadays, characterize the skyline of San Gimignano, the famous small town close to Siena.

An extensive study on the tower has been performed within the framework of the research contract called «San Gimignano Project». Research involved local Authorities (Municipality of San Gimignano, Tuscany Regional Administration) and four Departments of different Tuscany Universities (see Bartoli and Mennucci 2000). Even if all the towers have been studied in past years under several different points of view (mainly for architectural and archaeological investigation), no studies were performed in order to check the «safety» of the monument under a structural point of view.

Historical masonry buildings exhibiting a prevailing vertical character, such as towers and bell-towers, represent a structural typology with several common aspects: they are slender tall structures, which mainly have to support their own weight. These characteristics, together with all damages induced by several different factors during the years, make them particularly vulnerable with respect to (even small) base movements, such as those provoked by seismic actions or base settlements; the crack pattern which is inevitably present on these structures appear more or less typical of this kind of monuments.

Moreover, the evaluation of structural reliability of towers and similar structures is quite demanding: these structures possess a low safety margin with respect to external actions, because of the high level of stress induced by the self weight if compared with the ultimate resistance of the materials utilized in the construction. The masonry characteristics and the compression level are besides responsible for the very low ductile overall behaviour of the whole construction. In the past years, several examples of sudden collapse of important towers have been experienced: in 1989 the Torre Civica in Pavia felt down (Macchi 1993; Binda et al. 1992), while in 1993 a collapse interested the bell-tower of St. Magdalena Church in Goch. The collapse of the San Marco bell tower in Venice back in 1902 is another worldwide known example of a sudden tower failure.

The Torre Grossa in San Gimignano (Figure 1) is a 60 m height masonry tower, which presents a square cross section with a side of about 10 m; the masonry is constituted by two facing walls with filling material: the external wall is composed by 20–30 cm thick stone masonry made by prismatic blocks and minimal mortar joints; the internal wall is made by brick masonry with a thickness of about 25 cm; the core, with unknown mechanical properties, have a quite good cohesion (where it has been observed). The overall thickness of tower walls is about 2 m. The tower, built during XIII century, is located in Piazza Duomo (Square of the Cathedral), adjacent to the Palazzo Comunale (Town Hall) and the Cathedral.

Figure 1
The Torre Grossa in San Gimignano

The aim of the research work was to assess the structural safety of the tower, with respect to seismic actions mainly; at the same time, the goal of the research project was the tuning of a diagnosis procedure, which could be used also in the analysis of the other towers.

Multiple leaf masonry

The type of masonry constituting the tower (made up by three different layers: the external stone masonry, the in-fill material and the internal brick masonry) is often referred as a «multiple leaf» one: the overall mechanical behaviour is quite complex and it is not easily described by very simple models. As a matter of fact, the behaviour of the filling material (constituting the thicker layer) can be intermediate between two limit cases (see e.g. Egermann 1996).

In the first one, the whole structure could be considered as a «multi-layered material». Supposing that the core is composed by a material with sufficient binding, vertical actions can be distributed among the layers according to Hooke's law, then the bearing capacity depends on the relative stiffness of the layer themselves. In addiction to vertical stresses, horizontal stresses can arise in the two outer skins because of the lateral deformation of the in-fill due to the Poisson's effect.

In the second one, the in-fill can be treated as a material possessing no cohesive properties, due to the little or no binder content. In this case, vertical loads are supported only by the external skins, and horizontal loads arise at the interface layers between the core and the outer masonry, approaching the same limiting distribution typical of a «silo». Vertical loads are transferred from the core to the skins by the friction between materials, producing an increment in the vertical load to be supported by the external layers.

The real behaviour of a multiple leaf masonry is intermediate between the two limit cases; as a matter of fact the actual binding of the core material is neither absent nor stiff enough to ensure a full

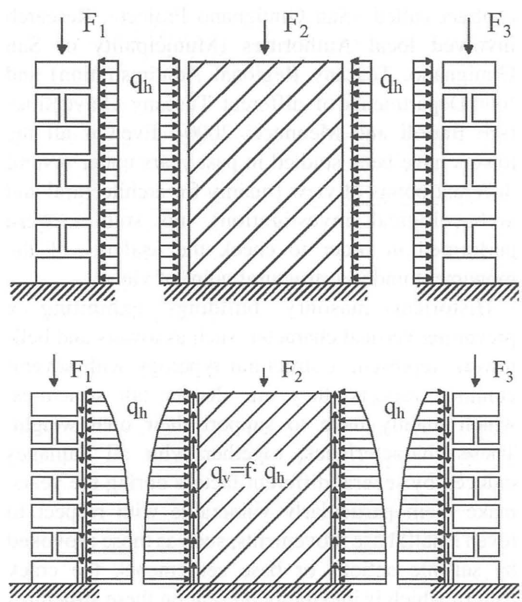

Figure 2
Multiple leaf masonry-Left: the masonry as a «multi-layered material»; Right: the masonry modelled as a «silo» containing a material with low cohesive properties

transmission of vertical loads. Moreover, a certain connection exists between the two layers: this can be either constituted by some brick layers or by horizontal tie-bars, so that ad additional mechanism could arise. Inside the in-fill, several «arch mechanisms» can be thought to be present, where horizontal ties are constituted by the connections between the two external layers, then reducing the horizontal action on the outer skins (see Figure 3).

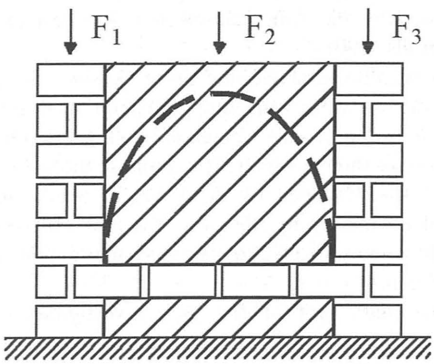

Figure 3
The «arch mechanism» inside a multiple leaf masonry

In a first step of the study, several numerical models have been set up in order to evaluate the stress state of the masonry under different loads. Then, an extensive experimental campaign has been performed in 1996, so as to better understand the actual behaviour of the monument. Differences between obtained results and those from the numerical model (especially for the stress state in the external stone masonry) required further experimental tests (performed in late 1999), mainly devoted to flat-jack tests on stone masonry as well as to get a clearer insight into the stone characteristics and the crack pattern affecting the tower.

Experimental tests

Experimental tests on the Torre Grossa consisted both in «in situ» tests (in order to identify the global structural behaviour and the local masonry characteristics) and in laboratory tests (to single out the mechanical characteristics of the tower constituting materials).

In the «in situ» experimental campaign both dynamical and mechanical tests have been performed.

Dynamical tests have been performed to evaluate the effects induced on the structure by some external actions, such as wind loading and earthquakes. Tests have been executed by using a vibrodyne placed at the top level of the tower, and recording the structural response by means of some seismic accelerometers and velocity transducers. The analysis of recorded signals allowed the identification of the main dynamic characteristics (natural frequencies and eigenmodes). Effects induced by bell movements have been recorded too (on the top of the tower one big bell is located, and its movement is strongly reflected on the structure).

Mechanical tests involved flat jack tests at several locations, so as to determine the local stress state as well as the mechanical characteristics of the masonry (mainly the Young' modulus by «double» flat jack tests); some samples have been also collected to individuate the filling characteristics.

Laboratory tests consisted in crushing tests on stone samples, in order to estimate the ultimate strength of the stone as well as its mechanical characteristics. 37 different samples have been tested, the ultimate strength varying from 43.35 MPa to 65.21 MPa.

All the obtained results have been used to define a numerical model, which was able to reproduce, as close as possible, the actual behaviour of the tower. The model has been built starting from results obtained from the geometrical and architectural survey of the monument, followed by a phase in which mechanical characteristics obtained from experimental tests have been assigned to the materials constituting the model. In the final step, the restraints of the tower have been identified, especially for the part of the tower adjacent to other buildings; the correct restrain level has been accepted when both main natural frequencies and eigenmodes in the model were correspondent to the recorded ones.

Flat jack tests

As it is well known, the flat jack test technique for the determination of both local stress state and stiffness properties in a masonry panel is based on the tensional release induced by horizontal cuts

performed on masonry walls. The test has been introduced starting about from 1979 and it is now widely employed. Main steps of the procedure are the following:

- placement of measurement bases;
- execution of the cut by means of a circular saw (usually within a mortar bed joint); the subtraction of resistant material causes a partial closure of the cut, inducing displacements accounted by the measurement bases;
- insertion of the flat jack into the cut;
- increasing of the oil pressure inside the jack until the original displacement state of the masonry measured before the cutting is recovered; in such conditions, the action of the flat jack reproduces the one previously given by the material removed by means of the cut; some interpretative parameter are defined in order to achieve the stress state in the masonry basing on the oil pressure inside the jack.

The stress acting before the cut is usually obtained following the expression:

$$\sigma = S_f = K_m \cdot K_a \cdot P_f \qquad (1)$$

being σ the stress acting before the cut, S_f the action of the jack, K_m a jack parameter depending on jack characteristics (geometry, stiffness, construction), K_a a second parameter depending on the ratio between the cut and the flat jack surfaces and P_f the jack internal oil pressure.

Distances between measurement bases are recorded initially, after the execution of the cut and following each increment of the pressure of the jack. Three or more measurement bases are usually employed (Figure 4), following the recommendations reported by the American Society for Testing and Materials (1991).

As mentioned above, the Torre Grossa is built by two facing walls with filling material and several pinning: due to the remarkable stiffness change among the three layers which compose the tower, the stress state induced by dead weight results to be sustained mainly by the stone external wall, which results to be subjected to a strong compression stress condition.

The main part of flat jack investigation have thereby been performed on the external stone masonry wall, in order of evaluating the actual compression stress state and, employing a couple of jacks, the overall Young modulus. Table 1 summarises some of the main obtained results by flat jack tests.

Table 1. Flat jack tests results

TEST # (year)	MASONRY TYPE (test type)	STRESS VALUES [MPa]	YOUNG' MODULUS [MPa]
1 (1996)	BRICK (double)	1.594	6130
2 (1996)	BRICK (double)	0.922	2270
3 (1996)	BRICK (single)	1.555	—
4 (1996)	STONE (double)	4.416	11530
5 (1996)	STONE (single)	4.416	—
6 (1999)	STONE (single)	4.800	—
6A (1999)	STONE (double)	4.800	11350
7 (1999)	BRICK (single)	0.864	—
8 (1999)	STONE (single)	5.568	—
9 (1999)	STONE (single)	6.720	—

In the examined case, the use of Eqn. (1) gave stress values in strong discordance with those from numerical analyses. In a second experimental phase, flat jack tests have been repeated, also evaluating the influence of their positions: in mortar bed joints or within a single stone block. In every case, a material behavior strongly different from the linear elastic one initially assumed was observed.

In order to achieve a better interpretation of masonry behavior, a particular test has been performed within a single stone block, so as to minimize effects from rigid rotations of other blocks or from the irregularities of joints surfaces (Figure 4). The placement of many measurement bases along the cut and the small pressure increments imposed during the load phase allowed an accurate description of the test, permitting a reliable interpretation of data. Figure 5 shows displacement curves obtained for the five measurement bases vs. the flat jack internal pressure.

The analysis of the obtained results reveal several relevant aspects concerning the behavior of masonry during the test:

- the initial displaced configuration and the following ones (correspondent to each pressure increment) result to be almost symmetric and much more regular with respect to the ones obtained from tests performed along bed joints;
- the incremental displaced configurations (i.e. the ones measured with respect to the configuration subsequent to the cutting) results to be represented with good reliability by two straight lines, showing thereby a very different behavior with respect to a linear elastic scheme (in this case, in fact, incremental displaced configurations would assume the same shape of the global displaced one);
- in correspondence of the recover pressure for the central base (no. 3 in Figure 5- about 56 bar), the extreme bases no. 1 and 5 present a recover of about 65% of the initial displacement: it is thereby evident the occurrence of localized (both elastic and inelastic) displacements along the cut boundaries.

To evaluate the results it is then necessary to abandon the linear elastic behavior scheme. A suitable model has been proposed by Bartoli, Chiostrini e Innocenti (2000) in order to take into account the nonlinear masonry behavior. The evolution of the strain state has been modelled by introducing an «equivalent» beam that, according to the crack evolution, possesses variable stiffness properties as a function of the applied load.

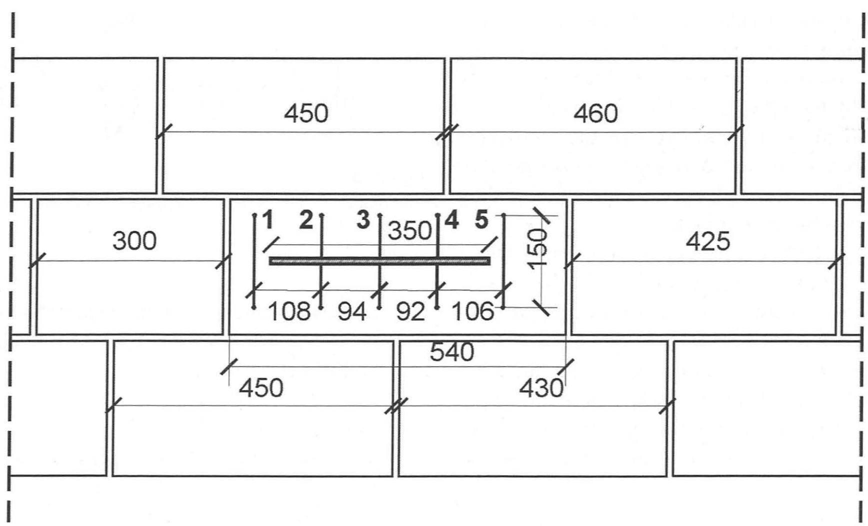

Figure 4
Base positioning [mm]

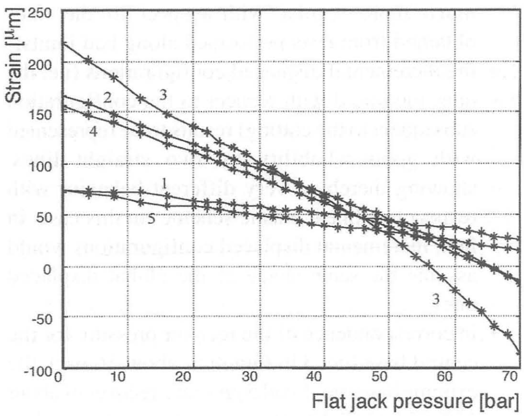

Figure 5
Strains vs. flat jack pressure for all measurement bases

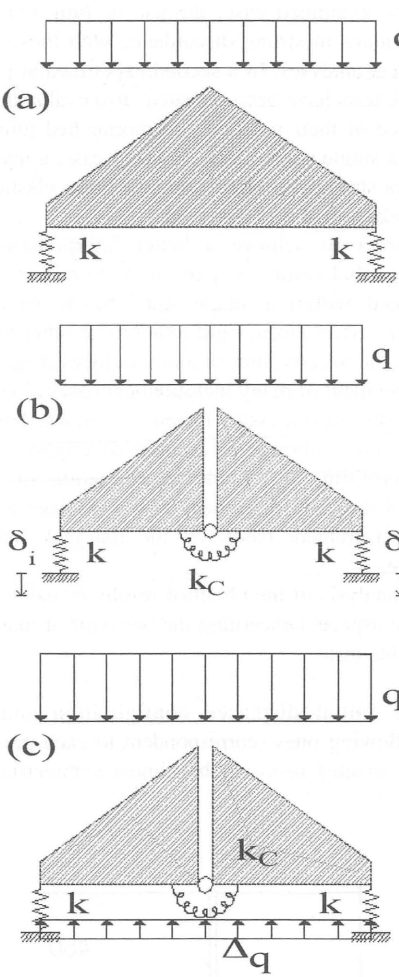

Figure 6
Equivalent beam evolution during the flat jack test

The beam (which has a trapezoidal shape) is elastically supported by the adjacent masonry; when a certain value of the stress has been exceeded in the central section of the beam (during the cutting procedure) the beam itself evolves to a different one, where an elastic hinge is located just at midspan, accounting for the stiffness reduction.

The condition of the beam described above (initially linear elastic, then formation of the hinge at midspan and inelastic displacements of supports) corresponds to the situation of the masonry sharply achieved with the execution of the cut. Effects of the load phase induced by the flat jack is finally represented by the upward load in Figure 6c.

Figure 7 depicts the load cycle of the equivalent beam in which a), b) and δ_i segments correspond to the cutting, while the c) phase represents the loading path performed by the flat jack.

Parameters of the equivalent beam defined above are: the material Young modulus E, the elastic stiffness of the end supports k, the elastic stiffness of the midspan pin k_C, the inelastic displacement of the end supports δ_i, q^* (formation of the middle span hinge) and q_{max} (actual vertical stress); all these values have to be estimated through experimental results. The main guidelines of the procedure are the following:

- k_C is determined from the incremental displaced configuration as the ratio between the bending moment at midspan of the beam and the relative angle between the two segments which form the configuration itself;
- k is determined referring to the numerical model of Figure 6a;
- E, q^*, q_{max} and δ_i are not independent and have to be calculated imposing the equivalence of experimental displacements from the flat jack test and the deformed configuration of the equivalent beam.

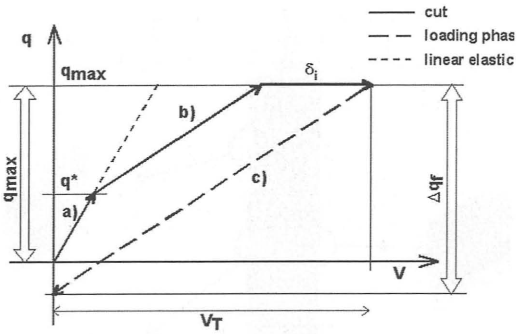

Figure 7
Load-displacement curve for the equivalent beam

The procedure has been utilized to process data from different flat jack tests, obtaining q_{max} values result substantially lower than $S_f = K_a \cdot K_m \cdot P_f$ from Eq. (1), confirming in this way that the cited formula leads to an overestimation of the actual stress state of masonry.

An additional «correction coefficient» β could then be introduced, accounting for the possible nonlinear behavior recorded during test

$$\sigma_{\text{actual}} = \beta \cdot \mathrm{S_f} = \beta \cdot K_m \cdot \mathrm{P_f} \quad (2)$$

Referring to Table 2, it is possible to conclude that in the case of the Torre Grossa in San Gimignano, the application of Eqn. (1) leads to an overestimation of about 20% with respect to the actual stress value in masonry walls.

Table 2. Comparison between results from Eqn. (1) and those from the proposed model

Test	Stress from Eqn. (1) $S_f=K_a \cdot K_m \cdot P_f$ [MPa]	Identified stress σ_{actual} [MPa]	$\beta = \frac{\sigma_{actual}}{K_a \cdot K_m \cdot P_f}$
Within a stone	6.72	5.50	0.81
Within a mortar joint	4.80	3.80	0.80

Determination of an «equivalent» stone

Three different stone types have been used to build the external wall of the Torre Grossa: a porous limestone, the travertine and a kind of limestone called «amphystegina». Travertine can be further subdivided into three different classes according to its porous' degree; in the following the three levels has been named as low porous, medium porous and high porous.

By laboratory tests, mechanical characteristics of all the different types of stone have been determined; Table 3 reports the obtained values.

Table 3. Mechanical characteristics of different stone types

Stone types	Young' modulus E [MPa]	Characteristic strength [MPa]
Porous limestone	45000	36.33
Travertine #1 (low porous)	35000	31.06
Travertine #2 (med. Porous)	33000	21.83
Travertine #3 (high porous)	12500	6.74
Limestone «amphystegina»	48000	32.66

Some researchers of the University of Siena have then proceeded to a localization of all the different types of stones on the North and South walls of the tower, so producing an accurate mapping of each single block; from the obtained results, it has been observed that travertine, the weakest among the used stone types, is predominant with respect to limestone, that limestone has been used in very localized part of the walls, and that, on average, the quality of the used stones is quite good (Dipartimento di Archeologia . . . 1997).

Starting from the obtained mapping, numerical models of some representative masonry panels have been set up. Each «numerical» panel has been modelled by using *Solid65* elements within those available in the finite elements code Ansys (Swanson Analysis System, Inc. 1996); this kind of element has been chosen because of the possibility of performing both cracking and plastic analyses. Each panel have

been restrained so as to reproduce a compressive laboratory test: nodes on the lower part have been fixed, while side nodes have been restrained in order to avoid any displacement in the transversal direction with respect to the applied vertical load. Upper level nodes have been restrained horizontally, while test has been simulated by imposing progressive vertical displacement and evaluating the force level within the panel; displacement increments have been maintained very low (0.5 mm for each loading steps) so as to perform an accurate step-by-step nonlinear analysis.

The aim of the numerical simulation was to individuate, according to different stone arrangement within the panel, several load-displacement curves, in order to define an «equivalent» stone, which could be used in the modelling of the whole structure.

Plastic behaviour of the materials has been described by means of the Drucker-Prager criterion; all the necessary parameters (cohesion, c, friction angle, ϕ) have been defined starting from uni-axial tensile strength f_t and compressive strength f_c.

Cracking of the materials have been introduced by means of the Willam-Warnke model (1974), where parameters have been tuned starting from uni-axial result.

All the introduced parameters have been then modified in order to take into account the actual materials' behaviour: as a matter of fact, the real behaviour is intermediate between a «purely frictional» one (characterized by a null value of the dilatancy, δ) and a «purely dilatative» one ($\delta = \phi$). Parameters' modifications are also necessary for taking into account the dissipative phenomena arising when cracking occurs; cracking, if no modifications are made, is in fact taken into account as a stiffness reduction only (Davis 1968; Chen 1975).

By means of the mapping of the South wall of the tower, ten different areas in which masonry was composed by travertine only have been singled out; the areas have been chosen so as to obtain an exhaustive description of all the main possible block combination in the stone masonry of the Torre Grossa (Figure 8). Each single area has then been numerically reproduced, determining its ultimate strengths; ultimate strengths have been thought as dependent on the percentage of different travertine types present in the panel as well as on their reciprocal position within the panel (Bartoli, Casamaggi and Spinelli 2000).

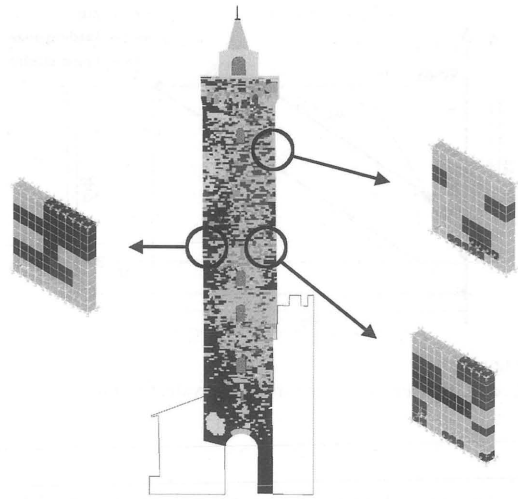

Figure 8
Mapping of travertine' stones and panels' numerical modelling (South side)

Once the ultimate strength has been determined for each panel, a suitable relationship has been found which allowed estimating the compressive strength of an «equivalent» stone able to represent the behaviour of the masonry, which constitutes the external wall of the tower.

A parameter R has been introduced, taking into account the percentage of the three different types of travertine within the panel. The parameter has been assumed as

$$R = a_1 R_1 + a_2 R_2 + a_3 R_3 \tag{3}$$

where R_1, R_2, R_3 are the ultimate strengths of low porous, medium porous and high porous travertine respectively, while a_1, a_2 and a_3 represent the respective percentages.

Once R values have been evaluated for each single panel, the same parameters has been evaluated for the whole tower (the tower being constituted by a 37% of low porous travertine, a 38% of medium porous travertine and a 15% of high porous travertine) and it has been called R_{tower}; R values have been plotted vs. the actual ultimate strength of each panel and some best fit approximation curves have been evaluated. The ultimate strength for the whole tower has then

been evaluated by using these approximation curves; the average between all the values obtained by intersecting the approximation curves with the R_{tower} value has then resulted to be equal to 9.31 MPa. The «equivalent» stone constituting the tower can therefore supposed to possess this value as ultimate strength and this one can be used as a reliable value for characterizing the materials used in the numerical model of the whole structure.

The survey of the crack pattern on the Torre Grossa

The investigation on the Torre Grossa allowed to point out the crack pattern on the external walls of the monument: the survey was possible thanks to the presence of a special moving scaffolding fixed on the North and South walls, which allowed to investigate the masonry at a very close distance. The two walls are affected by a complex system of cracks, while fewer cracks are also present in the other two external walls and in the internal brick masonry.

The procedure consisted in fixing on the external surface some measuring tapes in order to have a constant reference during all following investigations; as many pictures as those necessary to cover the whole surface of the tower have been taken from a distance of approximately one meter. The global surface has then been reproduced by joining adjacent pictures, so to have a complete reproduction of all the cracks; from the whole picture set, the full crack pattern on both the two investigated sides has finally been singled out (Figure 9).

The crack layout is in a good agreement with the one usually characterising masonry towers: main cracks are in the vertical direction, i.e. the same direction of main compressive stresses. The path followed by each single crack is obviously influenced by local masonry characteristics, such as mortar joints, which constitute preferred way for the cracks' development, even if, in several points, cracks pass through the stone blocks.

On the South side of the tower, a big fracture has been observed, and it is present in the internal brick masonry too; such a crack, presenting a maximum amplitude of about 1 cm, begins at the level of the upper window and goes down as far as the level of about +20 m with respect to Piazza del Duomo level.

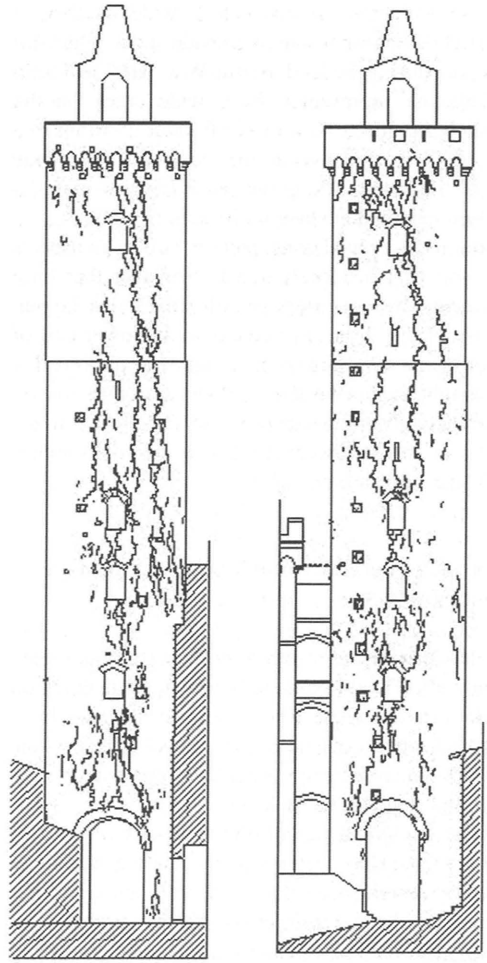

Figure 9
Crack pattern on South and North sides of the Torre Grossa

The crack is on the right part of the wall and, in its lowest part, splits into several minor cracks, ending in correspondence of the Palazzo Comunale, the historical building adjacent to the East side of tower. The presence of several cracks in its lowest part is maybe due a local stone crushing caused by the high compressive stresses.

On the North side, fewer cracks have been singled out, even if a larger diffuse damaging is present, caused by its orientation; in several points, joints appear opened, depending on the breaking up of the

mortar. Moreover, on this side a wide number of localized damaging is due to grenade shots, which hit the tower during Second World War. The two main fractures are represented by a wide crack on the central-right side from a level of about 25 m up to a level of 43 m, and by the one close to the left side edge of the tower. Along the tower corners, material has been expelled at several different heights.

From the surveyed crack pattern, two main aspects arise: the first is represented by the fact that base settlements are not responsible for the crack layout, because of the absence of cracks in the lower part of the tower; the second aspect is related to the fact that self weight cannot be the only cause of the cracks' occurrence, given that fractures are also present in the upper part of the tower, just below the small arches under the tower's battlement.

AN INTERPRETATION ON THE CAUSE OF THE MAIN CRACK PRESENCE

As a last part of the research work, a study has been performed on the possible cause of the main crack on the South side of the tower. Several analyses have been performed on a numerical model of the whole tower, looking for high vertical (compressive) and horizontal (tensile) stress levels.

In the numerical model, the thickness of the two masonry layers has been maintained as constant along the tower height; the stone masonry thickness has been assumed equal to 20 cm, while the internal brick masonry presents a thickness of 25 cm; the filling thickness has then been varied along the tower's height, according to the reduction of wall thickness. Main openings have been introduced in the model, together with all the cavities pointed out during the architectural survey; the consequences of the collapse of a wide part in the South-West edge due to a lightning which hit the tower in 1632 (the collapsed part was re-built in 1650) has been investigated in order to give an explanation to the cracks' presence.

The part collapsed after the lightning event has then been removed from the model; the stress pattern on the tower due to self-weight only appears substantially different with respect to the one when the collapsed part was still present, and vertical compressive stresses more or less follow the main crack layout. Stress levels are very high and are increasing toward the lower part of the tower; just below the zone restrained by the Palazzo Comunale, vertical stresses are lower and tend to be shifted toward the left part of the wall, where the survey pointed out a wide presence of small cracks, close to the main arch.

According to the vertical compressive stresses, horizontal tensile stresses arise, which could be responsible for the cracks' opening, so that, even in the North side of the tower, zones where highest compressive stress levels have been pointed out follow the main cracks' direction and location.

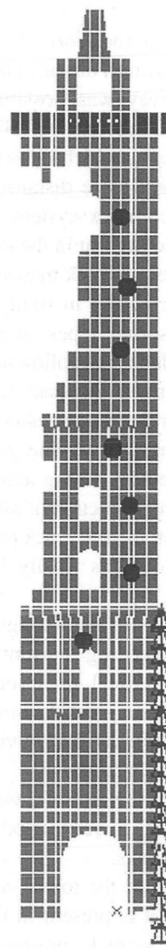

Figure 10
Position of vertical stress resultant at seven different levels (South side)

In order to confirm the hypothesis that lightning could be thought as responsible for the cracks' appearance on the tower, vertical stresses have been investigated by analysing the position of the vertical stress resultant on the South side of the tower at 7 different levels.

The position of the stress resultant at these levels has then been mapped on the side view of the tower, singling out the path followed by the cracks along the tower. As it can be seen from Figure 10, position of stress resultants at different levels matches very closely the shape of the main cracks, then confirming that lightning event played a fundamental role in the cracks' formation. It is quite interesting to note that, under the level of 17 m (which correspond to a zone below the level where the Palazzo Comunale is linked to the tower) the stress resultant is more or less centered with respect to the side of the tower.

It is also to be remarked dynamical effects due to the lightning as well as those derived from the collapse of the masonry should also be considered in the analysis, even if these two phenomena are very difficult to be modelled numerically.

Concluding remarks

In the paper, some of the main aspects related to a research work on the Torre Grossa in San Gimignano have been reported.

Nonlinear behavior of masonry has been observed performing flat jack tests on the wall external surface of the Torre Grossa, in San Gimignano. In order to obtain actual value of stress by means of such tests, an interpretation procedure has been defined, demonstrating that results obtained with the hypothesis of linear elastic behavior of masonry are overestimated of about 20% with respect to actual stress values (confirming otherwise results from numerical simulations obtained by F.E. models).

Starting from some experimental results, a numerical model has been set up, after a preliminary research work on the determination of the characteristics of an «equivalent» stone to be used in the analysis.

An extensive survey has been performed to identify the crack pattern on two of the four sides of the tower: a final numerical modelling confirmed that a lightning that hit the tower in 1632 was probably responsible for the cracks' appearance.

Acknowledgements

Authors wish to tank Mr. Saverio Giordano (Testing Laboratory of the Civil Engineering Department) for his precious help during the whole experimental campaign, and Dr. Alessandro Frati (Responsible of the Technical Staff of San Gimignano Municipality) for his assistance and cooperation during all the research work.

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Before 1695: The statics of arches between France and Italy

Antonio Becchi

A POST-DATED HISTORIOGRAPHY

The specialist literature indicates *Proposition 125* of Philippe de La Hire's *Traité de mécanique* (La Hire 1695) as the first contribution to the subject of the statics of arches, looked upon as a problem of mechanics applied to construction. It is well known that La Hire swept away the empirical rules known until then and which enabled the stability of an arch be determined on the basis of the width of the opening and, occasionally, of the height of its supporting piers. *Proposition 125* was followed by the formulations in terms of analogy between the equilibrium of an arch and that of a catenary (already guessed at by La Hire and investigated in further depth by Gregory, Bernoulli and Stirling) and by La Hire's own formulation dated 27th February 1712 (La Hire 1712), with the collapse analysis of an arch. Scholars are in agreement as to this linear historical genesis,[1] which focuses attention on the *Traité* of 1695, the turning point that overcame the intuitions of Leonardo[2] and the «building site rules» recalled by Gil de Hontañon, Martínez de Aranda, Derand and Blondel.

We wish to show here that the historical reconstruction referred to above ignores at least two very important texts: the comment to *Quaestio XVI*, contained in Bernardino Baldi's *In mechanica Aristotelis problemata exercitationes* (published posthumously in 1621) and the dissertation *Remarques sur l'épaisseur qu'on doit donner aux pieds droits des voutes et aux murs des dômes ou voutes de four*, read and delivered by La Hire to the *Académie d'Architecture de Paris* on 27th October 1692. These texts force us to review the assessments taken for granted up to now and to re-examine more closely the relationship between mechanics and architecture in the 16th and 17th century.[3]

BERNARDINO BALDI'S *EXERCITATIONES*

Bernardino Baldi (1553–1617) tackled the problem of arches in the course of an extensive and original comment to Aristotle's *Mechanical Problems* (Baldi 1621). Baldi's *Exercitationes* are not mentioned by La Hire, nor even by other authors who deal with the topic of arches in the 18th century, such as Danyzy, Frézier or Coulomb. Neglected by those concerned with *Construction History* and read inattentively by the historians of mechanics, it was probably the first printed text in which the subject of mechanics applied to architecture was tackled systematically and in which a clear configuration of the collapse mechanism of arches was suggested. This detail alone would be sufficient to make the *Exercitationes* very interesting, however there are at least two more aspects that should prompt a careful reading of the text: the originality of Baldi's approach to the Aristotelian *Problems*, which were discussed at length in the 16th and 17th centuries, and the singular way in which the treatise was developed with

reference to *resistentia solidorum*, which with great intuition linked *Quaestio XVI* to the subject of the solidity and of the thrust of arches.

> The Aristotelian *Quaestio* is well known. It is quoted here in the Latin translation (Baldi 1621, 95): «Dubitatur, quare, quò longiora sunt ligna, tanto imbecilliora fiant, et si tolluntur, inflectuntur magis: tametsi quod breve est ceu bicubìtum fuerit, tenue, quod verò cubitorum centum crassum?».[4]

It is helpful to remember that the *Quaestio* itself constituted the problematical backdrop of the Second Day of Galileo's *Discorsi e dimostrazioni matematiche* (1638), a text considered to be the basis for the whole of the great chapter of mechanics devoted to *resistentia solidorum*. It is referred to explicitly by Simplicio on the subject of solids having the same resistance, and the echo of that query can be heard throughout the Second Day, during which, however, the theory of arches was not mentioned. For Baldi, on the contrary, the Aristotelian posit was the natural starting point of an excursus which knows no equals in the albeit substantial bibliography referred to the *Mechanical Problems* and which tackled in twenty pages important problems of the statics of constructions. To find something similar, many years would have to pass and all the contributions scattered in a large number of dissertations drafted in the late 17th and early 19th centuries would have to be pieced together.

After tackling directly the problem of thin rods, which is the subject matter of the Aristotelian *Quaestio*, Baldi extended his analysis to topics of other kinds: a column bearing a weight and, generally speaking, the distribution of weight on a supporting surface; the collapse mechanism of the beam of a floor; the solidity of roof trusses and of lintels. The latter brings up spontaneously the problem of arches, to which approximately half the comment referred to *Quaestio XVI* was devoted, showing that he had a specific interest in this subject. For reasons of space it is not possible to tackle here the entire treatment provided by Baldi. This paper will therefore be limited to an analysis of the collapse mechanism (Baldi 1621, 112–114).

With reference to Figure 1, Baldi argued that a semi-circular arch ABC will tend to break following a divarication of the supporting piers and, consequently, of the two semi-arches AB and BC. Once this displacement has occurred it is possible to identify two stable parts, AQ and CR, in the semi-arches. These two stable parts correspond to a tripartition of the original arch featuring identical angles and which, together, form therefore two thirds of the complete arch. The stability of these elements is taken for granted by Baldi in an earlier passage (Baldi 1621, 109), in which the centres of gravity of the elements AQ and CR (Figure 2) are identified in D and H, on the perpendiculars to the supporting surface passing through A and C respectively.

The location of the *centrum gravitatis* on these perpendiculars subsequently enables Baldi to neglect the contribution towards thrust provided by the elements AQ and CR, so that he can concentrate instead on the central parts.

QB and BR tend to fall rotating around the intrados points Q and R (Figure 1). This rotation can be prevented in part if the distance QR does not exceed

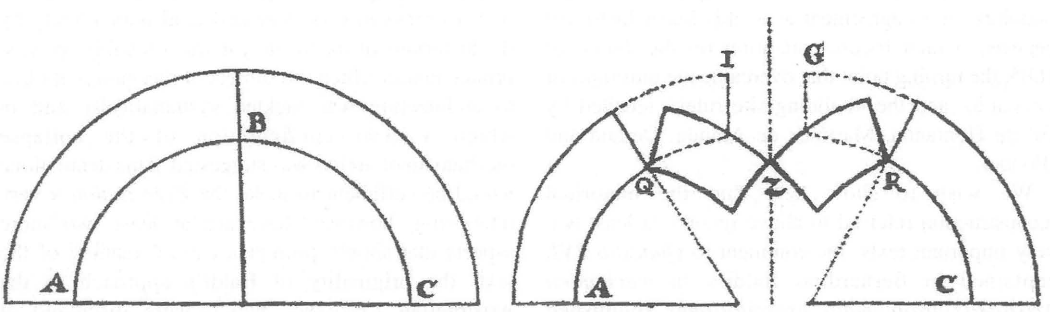

Figure 1
From (Baldi 1621), redrafted

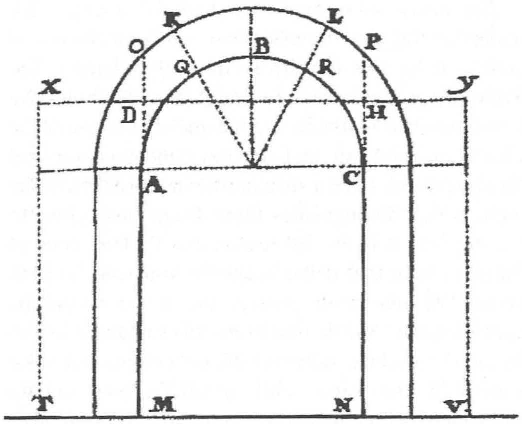

Figure 2
From (Baldi 1621), redrafted

the sum of the segments QI and RG, and it reaches a limit position when the two vertices I and G meet at point Z. According to Baldi, this collapse mechanism shows why thicker arches are more solid: indeed, in case of a thicker arch, full rotation of the central elements will only be possible in the event of a divarication of the imposts greater than that required for the previously discussed arch.

It is easy to see that the mechanical aspect of this «demonstration» can hardly be agreed with, but it does highlight three important aspects of the issue, that will crop up constantly in discussions on the mechanics of arches and vaults:

1. the tripartition of the arch enables two stable parts of the arch and one unstable central part, in which the collapse mechanism is triggered off, to be identified.
2. The central part does not form a single body delimited by the joint planes that separate the stable part from the unstable part; instead, it is separated into two parts along the keystone line.
3. The two central parts, split up as stated, do not slide along the breaking joints but rotate around the intrados edges.

In Bernardino Baldi's argument it is possible to glimpse one of the reasons that must have given rise to the «empirical» rule suggesting that the intrados of the arch should be divided into three equal chords in order to be able to determine the thickness of the supporting piers (the so-called Derand's rule, not mentioned in the *Exercitationes*), while the complete absence of explicit considerations referred to friction between the parts can be noted. The collapse mechanism with rotation of the elements that are considered unstable, on the other hand, becomes very important. It must be noted that the conclusive statement, concerning the advantage of having thicker vaults, does not prevent Baldi from illustrating the building custom according to which it was suggested that vaults should be made lighter in the central part and that the space above the springers should be filled.

Philippe de La Hire was to dwell on this aspect many years later, tackling the problem of arches starting out from completely different assumptions.

Proposition 125 and the *Mémoire* of 27th October 1692

The *Proposition*, and consequently the *Traité* containing it, has always been analysed without referring in any way to the architectural context of the time, but if the reasons which induced La Hire to tackle this subject are considered in detail, it is possible to find many clues that necessarily change the point of view from which the work is seen. The arguments contained in the *Traité* lead clearly to the activity he carried on in the framework of the Paris *Académie d'Architecture* [5] and to his interest in *la coupe des pierres*, which are linked to the direct relationship he had with Desargues and with the unpublished work *Traité de la coupe des pierres*[6] (La Hire 1687–1690).

> *Proposition 125* must be viewed as the natural continuation of the discussions which ensued in the Academy following Leon Battista Alberti's *De re aedificatoria* reading. The minutes of these meetings are precise and circumstantial in this respect (Lemonnier 1911–1929).[7] On 20th October 1692 the academics (the *Compagnie*) commented the pages of Alberti's Treatise dealing with how thick the walls of round temples should be (Alberti 1485, book VII, chapter X): «Il dit que dans les temples ronds que l'on veut rendre fort solides, on doit donner aux murailles la moitié du demi diamètre intérieur du temple, ce que l'on a approuvé, en se réservant néantmoins d'en parler encore la première fois et de faire

quelques mémoires sur ces sortes de proportions». On 27th October, at the next session, La Hire read a *mémoire* to the *Compagnie* containing the observations developed by him on that same subject and which was to be commented on again at the time of the next session, held on 4th November.

The text attached to the minutes of 27th October resembles a draft of *Proposition 125*, however it also contains more structured thoughts, which consider in greater depth some aspects of the problem that were to be completely left out of the *Traité de mécanique*. Indeed, although the paper was still immature from the point of view of mechanics, was extremely explicit from that of construction. In it, La Hire situated the architectural problem and, above all, clarified his thoughts on the mechanical behaviour of arches and on the relevant *règles de l'art*. The *mémoire* submitted to the *Académie* and approved by the latter bears the title *Remarques sur l'époisseur qu'on doit donner aux pieds droits des voutes et aux murs des dômes ou voutes de four*, and started out from Alberti's proportions (La Hire 1692). La Hire proposed a comparison between the dimensions indicated by Alberti and those resulting from Derand's rule, and demonstrated that if the vault is full-centred the two indications coincide. He pointed out, however, that «cette règle ne peut estre fondée que sur quelques expériences», as is obvious if the piers considered are very high in relation to the size of the vault. Assuming, for instance, that the height of the columns is equal to one and a half times the diameter of the vault, the indications provided by that rule would turn out to be unacceptable.[8]

Before proceeding with his mechanical analysis, which has the intention of overcoming the fragile empirical nature of Derand's rule, La Hire expresses a decisive premise, which guides everything that follows it: if, in a stone vault, «tous les voussoirs ettoient tous poussez vers le centre de la voûte avec un mesme effort», then this vault would not thrust against the supporting piers «car la clef et les voussoirs d'en haut qui en sont proches ne feroient pas plus d'effort que s'ils estoient tous joints ensemble et s'ils ne faisoient qu'un mesme solides, dont il faudroit considérer l'effort comme celui d'une seule pierre toute droite pesant autant que tous les voussoirs ensemble et posée de niveau sur les pieds droits».

The observation expressed here is essential for understanding the first *mémoires* on the mechanics of arches. It heralds the theory developed later in the *Traité de mécanique* —to gauge the weight of the voussoirs so as to obtain «un mesme effort»— and the idea is pointed out that the keystone voussoir and those next to it have a special role in the statics of the arch, which distinguishes them from those close to the imposts. It is for this reason that La Hire goes so far as to state that if the «central» voussoirs were to consist of *une seule pierre*, the thrust would be cancelled out. A little further on this statement was to be altered slightly, however the conceptual reference remained the same and recalled that already expressed by La Hire at the meeting of 19th November 1688 about the *coupe des pierres* in «voûtes surbaissées».[9]

Proceeding in his analysis, with his attention focused on the single voussoirs, La Hire continues to refer to a 'non-thrusting' construction and the *monolithic model* constantly influences the reasoning he follows. This same consideration inspired the problem included in *Proposition 125*, that is to say, «donner une règle pour faire que les premiers voussoirs récompensent par leur pesanteur ou par leur charge l'effort de ceux qui sont vers la clef». Again in this case the attention was focused, as revealed in the title, «sur l'effort» produced by the voussoirs close to the keystone and not by all the voussoirs undifferentiatedly. The demonstration given in 1692 was altered in the *Traité* (1695), however not only did the basic idea remain the same on that occasion but, as we will see, it would also condition the *mémoire* of 1712, more than the text of *Proposition 125* did. In this latter work, furthermore, there was nothing to bring to mind a profound knowledge of a building site and of proper workmanship, with regard to which La Hire had simply reiterated well-known concepts.

The rules of proper workmanship do, on the contrary, play an essential part in the 1692 *mémoire*, in which an explanation according to «les principes de la Mécanique» is attempted. La Hire states that his mechanical considerations are clearly confirmed in site practices, in that the custom of loading the voussoirs close to the springers definitely confirms the validity of those observations, since experience had shown that this construction practice made the vaults more solid and safer.[10] At the end of the *mémoire,* this concept is reasserted in even stronger

terms. Since it had been demonstrated[11] that the keystone and the voussoirs that are close to it push against the abutments far more than do the other voussoirs, then it would be sufficient to remove the keystone and a few adjacent voussoirs for this thrust to be considerably lowered. The voussoirs close to the imposts would then require very little weight in order to withstand the pressure with which the others tend to press against them. Again in this case, on-site experience tended to confirm the theoretical reflection referred to the construction of dome: «Il est donc certain que l'usage d'ouvrir les dômes vers le milieu, comme pour y mettre une lanterne, soulage beaucoup la voûte et empesche l'effort des voussoirs à écarter les murs et piliers buttans».

The procedure that leads to the definition of the weight of the single voussoirs obviously comes up against the problem of the first voussoir of the impost, as was to be the case in *Proposition 125*. If the voussoirs are «infiniment polis, en sorte qu'ils peuvent glisser les uns sur les autres sans aucune difficulté», as is stated in the foreword, then there is no weight that could enable that voussoir to withstand the effort transmitted to it from those above. Once again, here, La Hire introduces a reflection drawn from his experience of construction: «C'est pourquoy on devroit arrester soigneusement ce voussoir avec le coussinet pour faire une bonne construction, si les inégalitez des pierres ne l'empeschoit de glisser sur le coussinet, et se sont aussi ces mesmes inégalitez qui récompensent en quelque façon les grandes charges qu'il faudroit donner à tous les voussoirs et surtout aux premiers».

What is today called friction came into La Hire's reflection, albeit in a still indeterminate manner, as the effect of the *inégalitez des pierres*. In this respect, it is important to note the difference as compared with *Proposition 125*, in which, on the other hand, «matiere qu'on met entre deux» was mentioned, stressing the importance of the presence of mortars and moving away from the world of *coupe des pierres*; if for no other reason then in order to respect the initial hypothesis regarding *infiniment polis* voussoirs, on which doubt could not be shed in a *Traité de mécanique* simply by means of a vague reference to experience.

In order to understand the context of the *mémoire* written in 1692 better, some considerations of a general nature should be added. The *monolithic model* from which La Hire started out had a clear precedent in stereotomy, and the assumption on which the *coupe des pierres* was based must be seen in the idea of a whole made up of *une seule pierre*. At the beginning of the *Traité de la coupe des pierres*, drafted five years earlier, La Hire had written the following, interpreting the entire stereotomic tradition in this way: «les ouvriers appellent la science du trait dans la coupe des pierres, celle qui enseigne à tailler et à former séparément plusieurs pierres, en telle sorte qu'étant jointes toutes ensemble dans l'ordre qui est leur convenable, elles ne composent qu'un massif qu'on peut considérer comme une seule pierre».[12]

The same concept was reiterated on other occasions, for example on 11th January 1694 on the subject of the drums of columns,[13] and the reflection on the subject of *comme une seule pierre* was subsequently to become the underlying theme of other research studies into the thrust of vaults.[14] The monolithic idea was a mainstay of stereotomic art, precisely because it was founded on the need to make up the all with the parts, to create the whole with the discreet. This monolithic nature could be realised ideally by perfecting the rules of workmanship but also by providing the additional solidity that was attributed to cramps or to the wedges, often dovetailed in shape, that were positioned between one voussoir and the next. The same solution is often also found in ancient and medieval architecture[15] and became a matter for discussion in treatises, as shown by De l'Orme's treatise (De l'Orme 1567).

La Hire himself discussed a similar issue in *Architecture civile* (La Hire 1698) and, only a few years after the *mémoire* analysed above, in his *Projet d'une nouvelle construction de murs de brique et de pierre de taille*, read and approved by the *Académie d'Architecture* on 14th September 1699, in which a new building system calling for the use of bricks was presented.[16] The same subject was treated by Jean Errard, who illustrated his *Premier livre des instruments mathematiques mechaniques* (Errard 1584) with very eloquent plates.

Without defining them, La Hire postulated two types of monolithicity, in addition to the type on which the *coupe des pierres* is based. These were based on the specific arrangement of the joints, and were an *acquired monolithicity*, featuring the use of cramps or wedges to connect the voussoirs to one another, and a *monolithicity secundum situm*,[17] due to the «liaison [. . .] de leur propre

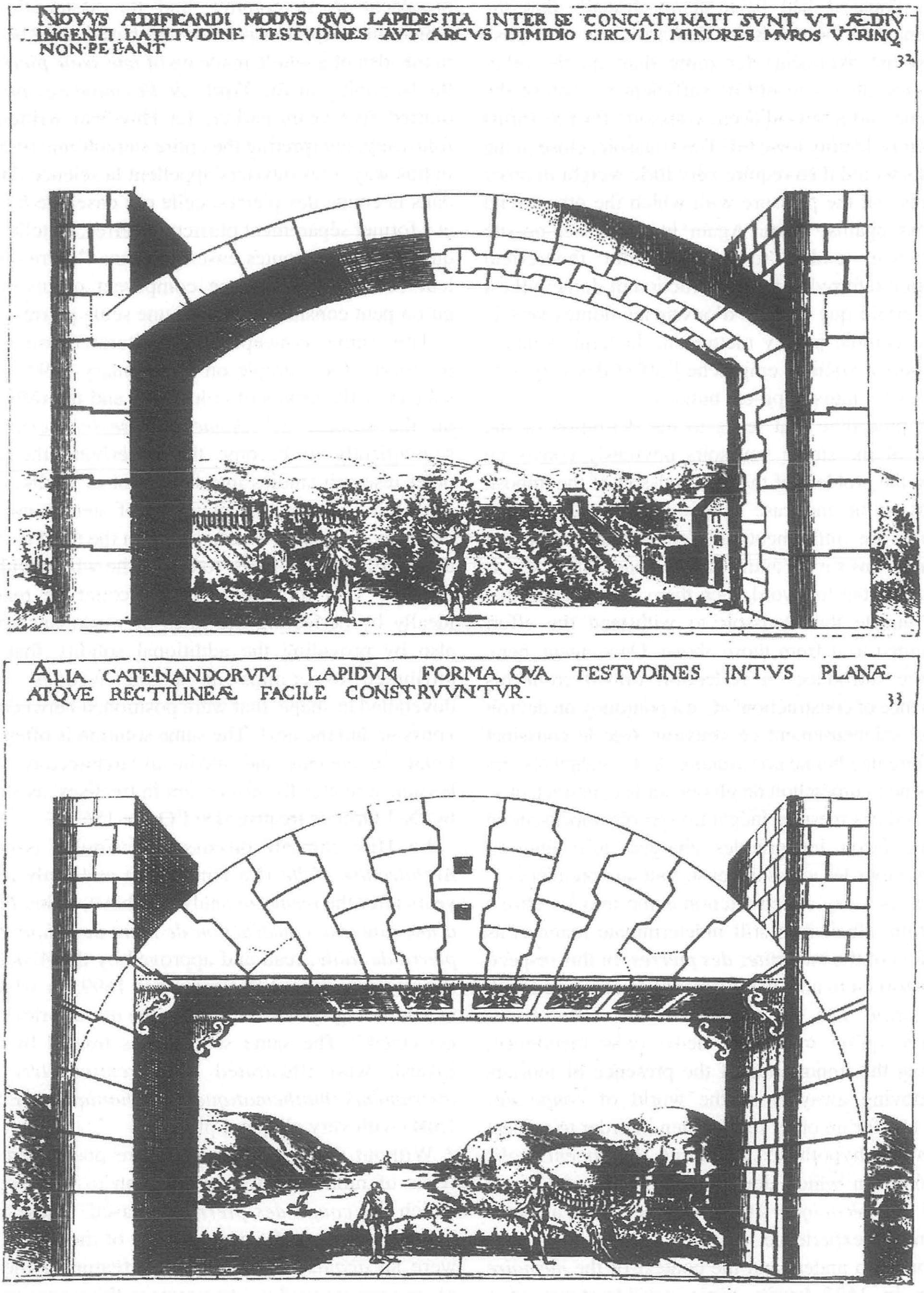

Figure 3
From (Errard 1584)

pésanteur»,[18] originating from on the theorem illustrated in 1692 and perfected in the *Traité de mécanique*. In both cases, the analysis was still developed in the framework of an investigation into the *force des voûtes,* and its stated purpose was to create a *non-thrusting* structure. In the first case the result depended on the effectiveness of the connections[19] and in the second on the «inegalitez des pierres», invoked in the 1692 *mémoire*, or on the «matiere qu'on met entre deux» mentioned in *Proposition 125*.

The considerations connected with the 1692 *mémoire* become even more eloquent on re-reading the one submitted to the *Académie des Sciences* on 27th February 1712. Again in this case there is good reason to believe that this text is similar to that discussed at the *Académie d'Architecture* during its session of 20th June 1711.[20] The same theoretical goal was considered from a different perspective, and the influence of the discussions held at the *Académie d'Architecture* could be felt yet again.

From 27th October 1692 to 27th February 1712

The 1712 text has been commented on extensively by many authors and rather than being necessary to re-examine it in full, it is sufficient here to pause to consider the initial hypothesis that conditioned the way in which it unfolds.

After noting that in architectural works the size of abutments varied from the excessive dimensions imposed by builders who were «moins hardis» and the insufficient dimensions due to the «trop hardis», La Hire states that «on remarque ordinairement que lorsque les pieds-droits d'une voûte sont trop foibles pour en soutenir la poussée, la voûte se fend vers le milieu entre son imposts et le milieu de la clef».[21] Starting out from this statement, apparently drawn from his experience and concerning the position of the breaking joint at an angle of 45° to the impost line (in the case of a semi-circular arch), La Hire deduces that «on peut supposer» that in the upper half of the semi-arches all the voussoirs are so well bonded to one another «qu'ils ne forment qu'une seule pierre». The whole subsequent reasoning for calculating how thick the direct bearing would have to be in order to guarantee the equilibrium of the arch is based on this supposition, taking the solidity of the foundations of these abutments for granted.

Commentators point out this hypothesis as an error, and do not analyse its contents.[22] The error, however, conceals much knowledge deserving of attention. It is obvious that the point from which the new theory started out coincided with the conclusions presented in the 1692 *mémoire*. The initial hypothesis is nothing other than a refined reproposition of what had already been written in this latter *mémoire* and, in particular, of the considerations on construction that had characterised it and that had disappeared in the *Traité de mécanique*: the voussoirs close to the keystone are the ones that produce the strongest thrust while those close to the imposts are integral with the supporting piers, also because of the «inegalitez des pierres». For this reason, La Hire had written that the central parts of domes could be made lighter in order to reduce the thrust on the walls. We should add that the load on the springers, which was recommended on that occasion, could give even greater reason to believe that a vault loaded in this way, in accordance with the rules of proper workmanship, would fail first of all at the point marking the upper boundary of the extrados area which was filled in.

Other references, however, came into the reasoning suggested by La Hire, although not in an explicit manner. In the *Académie*'s reports, the problem of the thrust of vaults was frequently associated with that of the thrust of soil and the correspondence between these two research areas was to be reaffirmed subsequently elsewhere, for example by Pierre Couplet at the beginning of his essay *De la poussée des voûtes*, read to the *Académie des Sciences* on 9th February 1729.[23] This link was considered entirely natural, and Pierre Bullet, in particular, had dwelt on these two aspects. As far back as 1686 he had treated the thrust of soil at the *Académie d'Architecture*[24] and on this topic he had later proposed his own theory in *Architecture pratique* (Bullet 1691), presented to the *Compagnie* on several different occasions. In this text it was explained that the angle of 60° must be considered the angle of natural slope, but also that it is preferable in calculations to refer to an angle of 45°, «pour tenir sur cela le chemin le plus seur» (Bullet 1691, 171). It does not seem improbable that La Hire would have taken this into account with reference to the possibility of the voussoirs remaining in equilibrium on the surface of the joint.

A second indication must be added to this, and it comes from a field of investigation that, although it

apparently has little to do with the *poussée des voûtes*, is actually connected directly with the topic at issue. In 1699 Guillaume Amontons had submitted a project for a «Moulin a feu»[25] to the *Académie des Sciences*, and had pointed out, with reference to this new invention, the problems of friction that condition the movements of this type of equipment. One observation, in particular, had baffled the academics, as it was in apparent contradiction with common sense (Amontons 1699a, 166): «Par ces experiences on peut remarquer, en passant, que c'est une erreur de croire, que les frottemens dans les machines augmentent ou diminuent à proportions que les parties qui frottent, ont plus ou moins d'étendue, et que la roue par exemple d'un moulin tourne d'autant plus facilement, que ses tourillons ont moins de longueur, ce qui d'ailleurs est une mauvaise construction, à cause qu'ils mangent incontinent les boëtes dans quoi ils tournent».

A lively debate on the truthfulness of this statement had immediately ensued at the *Académie,* and La Hire, who was an authoritative member of the *Académie des Sciences*, had carried out several experiments in order to clarify the terms of the issue. To this end he had conducted tests with samples of wood and marble, going so far as to confirm the independence of friction from the size of the area of contact and to explain those cases which could not be referred back to Amontons's intuition (*Histoire de l'Académie* 1699, 128–134). La Hire was therefore well aware of the phenomena arising out of *frottement*, that is to say out of the *inegalitez des pierres* of which he spoke in his *mémoire* in 1692 and which Amontons was to analyse, using exactly the same terms, in 1699.[26]

If we add to the *mémoire* of 1712 the references to the *coupe des pierres*, to the problems connected with the thrust of soil and to those arising out of friction, La Hire's hypothesis becomes easily comprehensible. It can be believed that the breaking point of an arch is located in the proximity of the angle beyond which friction is no longer capable of ensuring equilibrium (as had already been suggested in the *Remarques sur l'époisseur*) and that this is at approximately 45° to the line defined by the imposts.

This interpretation was confirmed in full by the assertions of Amédée François Frézier, who had attended La Hire's lessons at the *Académie d'Architecture*, in his *Traité de stéréotomie*. In this treatise he wrote that, thanks to friction, the voussoirs of the vault do not slide over each other until an angle of about 22° «et même jusqu'à 25. dégrez» is reached. He also adds that even beyond this angle, up to 45°, they produce very little thrust, «puisque ce n'est qu'à cette hauteur que les Voutes se fendent». (Frézier 1737–1739, 3: 397). This is consistent with what La Hire had asserted in 1692 and reiterated in 1712.

The myth of Galileo

There is a profound difference between the mechanical reasoning followed by La Hire and the approach suggested by Baldi, and this is explained at least partly by the cultural contexts from which they originated. On the one hand there were the world of construction of the *architecture à la française* and the experimental research promoted by the *Académie des Sciences* in Paris, and on the other mechanics in the tradition of Aristotle and the overview of Italian brickwork architecture, in which *coupe des pierres* never played a leading role comparable to the one it had in France.

In La Hire's analysis, the study of the effect leading to an analysis of the cause was still influenced by a steretomic approach, which indicated the pathway to be followed: the stones at the top, comprised between the two breaking joints, behave like a single *voussoir* and the kinematics underlying the interpretation of this concern the large keystone voussoir which pushes against the springers of the vault, as already described in the foreword to the 1692 *mémoire*. The mechanism being analysed is that of the wedge, on which the whole issue of *coupe des pierres* rests, albeit without drawing any strict mechanical consequences. It is highly probable, therefore, that the error with regard to the collapse mechanism was caused not only by the fact that «on remarque ordinairement» as suggested by La Hire, but also by the stereotomic principle according to which a well-built vault would behave «comme une seule pierre». Having verified break in a particular joint, it was easy to reiterate the principle, provided of course the vaults were built with proper workmanship, possibly with the arrangements described by De l'Orme and discussed by the *Académie d'Architecture*. The prejudice referred to the monolithic nature of the central part of the arch,

which in Jacques Heyman's essay had already been indicated with lightning intuition as the «'monolithic' approach» (Heyman 1976, 30), thus corresponds in full. On viewing the collapse mechanism, it is possible to speak with good reason of a *monolithic paradigm*, handed down from the *coupe des pierres*, with an implicit but probably decisive influence of the research being conducted at that time into the thrust of soil and into the role of friction.

La Hire's reasoning, frequently described as naïve and summary, was in actual fact perfectly pertinent to the scientific and technical context in which he worked and, above all, it was consistent with the world of construction that La Hire was required to consider in the framework of the *Académie d'Architecture*. The arches and the vaults he imagined in the abstraction of the *art du trait*, but had also observed in the field during his many travels all over France, corresponded adequately to this model. The lever and the wedge, considered «simple machines» that become the rules of grammar for mechanical interpretation, were grafted onto a conceptual context with which La Hire was particularly familiar. The context in question highlighted the limits of a strictly mechanical approach and, at the same time, revealed the ties to that *art de bâtir* which it was, in any case, necessary to confront. *Acquired monolithicity* and *monolithicity secundum situm* were an unmistakable sign of this *internal* interesse in stereotomy, and the activity carried on at the *Académie d'Architecture* appears to be something more than a mere pretext.

In the collapse mechanism proposed by Baldi, on the other hand, the presence of studies of a different nature and an ill-concealed discomfort, shared moreover by La Hire, can be felt on linking the principles of mechanics known at the time with on-site experience. The latter is mentioned explicitly and illustrated with pictures in the *Exercitationes*, which can hardly be accused of an exclusive preference for the theoretical component of the problem. This was, on the other hand, to occur subsequently (markedly during the course of the 19th century). The difficulty lies, rather, in finding a way to reconcile the mechanical situation described by the principle of the lever and by the *scientia de ponderibus* with the situation brought to the forefront by the arch, imposing compliance with analytical procedures that are today part of the basics of mechanics applied to construction but which at that time had still to be described.

The absence of considerations on the theory of arches in Galileo's *Discorsi e dimostrazioni matematiche* —although he definitely recalled the *Quaestio XVI*— and the limits of his analysis of the mechanical behaviour of a cantilever beam —in which the curvature of the inflexed beam and, therefore, the preconditions for the analogy between beam theory and that of elastic curves are neglected— highlight a technical and scientific context *Before 1695* rich in subtleties that could hold many historiographic surprises in store. Researchers interested in the relationship *between mechanics and architecture* have the task of accustoming their gaze to the dazzling light of the myth of Galileo and of setting to work to re-write some parts of the *Histories* that up to now have been considered as *Reference Works*.

NOTES

1. See the treatises mentioned in the references listed in (Becchi and Foce 2002). In (Benvenuto 1991) attention is drawn to some interesting pages by Honoré Fabri (Fabri 1669), however on the whole this approach is shared. Indeed, chapter 10, *First Theories about the Statics of Arches and Domes* (Benvenuto 1991, 2: 321–348), opens with an analysis of de La Hire's *Traité de mécanique*.
2. As yet there has been no thorough study of Leonardo's manuscripts dealing with this issue. Leonardo's writings are always viewed in a rather impromptu and non-systematic manner, as if his research into a given topic mirrored the fragmentary and asystematic nature of the subject-matter of the study.
3. For a fuller comment on de La Hire's *mémoire* see (Becchi 2002). The Author is currently working on a study of Bernardino Baldi's *Exercitationes*, to be published shortly with the title Quaestio XVI. *Dai* Mechanica *aristotelici alla meccanica per l'architettura: il contributo di Bernardino Baldi*.
4. «Why are pieces of timber weaker the longer they are, and why do they bend more easily when raised; even if the short piece is for instance two cubits and light, while the long piece of a hundred cubits is thick?»
5. De La Hire became a member of the *Académie d'Architecture* on 7th January 1687, to replace François Blondel, who had died in the previous year.
6. There are at least five copies of the manuscript, at the *Bibliothèque de l'Institut* and at the *Bibliothèque de l'École Nationale des Ponts et Chaussées* (this latter library has two copies of the manuscript) in Paris, at the

Bibliothèque municipale in Rennes and at the *Bibliothèque municipale* in Langres. See (Becchi and Foce 2002).

7. From now on only the date of the minutes will be mentioned.
8. (La Hire 1692): «Cette règle ne peut estre fondée que sur quelques expériences, car on ne peut pas assurer qu'un arc de pierre par exemple de 12 toises de diamètre, dont les pieds droits seront de 3 toises d'époisseur, soit si ferme qu'il n'ait pas besoin de piliers buttans ou de culée pour l'entretenir. Au contraire, il est très certain que les voussoirs feront toujours assez d'effort pour écarter les pieds droits, surtout s'ils sont d'une hauteur considérable comme d'une fois et demie le diamètre de la voûte».
9. «M. de la Hire a apporté à la Compagnie une démonstration dans laquelle il fait voir que, dans les voûtes surbaissées, la clef et les autres voussoirs qui en sont proches font plus d'effort pour escarter les premiers voussoirs que ceux-cy n'ont de force pour y résister, ce qui se prouve par la proprieté du coin, qui est plus aigu dans la clef et dans les voussoirs qui lui sont proches que dans les autres. Il y auroit plus de solidité si tous les joints de lit tendoient au centre de l'ovale qui forme le cintre surbaissé; mais cela n'est pas si agréable à la veue, cependant on est obligé de tomber en ce cas en plusieurs rencontres». This statement is significant, since it reverts to a consideration that had already been introduced in (La Hire 1687–1690) and makes it more explicit in the direction that was to be developed four years later (La Hire 1692).
10. (La Hire 1692): «c'est pourquoy les voûtes donts les reins sont bien remplis ont toujours plus de solidité et de fermeté que les autres».
11. (La Hire 1692): «On peut voir par la proportion que je viens de trouver que la clef et les voussoirs qui en sont proches font un bien plus grand effort dans une voûte pour écarter les pieds droits que les autres qui sont vers le coussinet».
12. (La Hire 1687–1690), sheet 1. Passage also contained in (Pérouse de Montclos 1982, 85), but with a transcription error : «pièce» instead of «pierre».
13. See minutes of 11th January 1694, *mémoire* bearing the title *Nouvelle manière de former des colonnes par tambours*: «Je crois que la meilleure de toutes les manières dont on puisse se servir pour poser les pierres, c'est de frotter les lits les une contre les autres avec un peu de grés et d'eau et de les arrester ensuite à la place où ils doivent demeurer. Car, par ce moyen, ces pierres se touchant exactement par leurs lits et ne pouvant pas s'approcher plus d'un costé que d'autre, ne forment que comme une seule pierre, et les arrestes des joints ne sçauroient s'éclater pour quelque charge qu'on élève au dessus».
14. See (Frézier 1737–1739, 3: 382): «M. Danyzy fit ensuite voir par une expérience que plus la clef est large moins la poussée de la Voute est grande: car si l'on substitue à trois ou à plusieurs Voussoirs une seule clef qui occupe tout l'intervale qu'ils remplissoient, et qui soit égale à leur somme, on verra que la Voute qui n'auroit pû se soûtenir après avoir un peu diminué de la force des piédroits, se soûtiendra cependant encore lorsqu'on y aura mis cette clef, quoiqu'elle soit aussi pésante que l'étoient les Voussoirs, non dans l'état d'équilibre, mais lorsqu'ils surpassoient la résistance des piédroits. D'ou l'on tire naturellement une conséquence que nous avons établie ci-devant pour une chose constante, que si la Voute étoit toute d'une pièce, la poussée déviendroit nulle». See also (Danyzy 1732, 52) and (Cosseron de Villenoisy 1869).
15. On this topic see (Reveyron 1996).
16. In the *mémoire* it is stated that: «Les anciens architectes ont pris de très grands soins pour lier toutes les pierres qui formoient les gros murs des édifices considérables, et nous voyons dans ceux qui sont batis de gros quartiers de marbre que toutes les pierres sont attachées les unes aux autres avec des clous et des harpons de bronze. Aussi ces édifices, après un grand nombres de siècles, sont aussi entiers que s'ils étoient nouvellement construits». (see attachment to the minutes of 14th September 1699).
17. We suggest this definition with obvious reference to the *gravitas secundum situm* described by Jordanus de Nemore.
18. According to the beautiful expressed used by Frézier. See (Frézier 1737–1739, 1: vii-viii): «Il faut en effet plus d'industrie qu'on pense pour que [les petites parties] soient [. . .] disposés de manière qu'elles se soutiennent en l'air, en s'appuyant réciproquement les unes sur les autres, sans autre liaison que celle de leur propre pésanteur».
19. There is a precise filiation, on which it is not possible to dwell here for reasons of space, between the *acquired monolithicity* investigated by de La Hire and the investigation of *linteaux armés*. On this latter subject, see (Saddy 1987) and (Middleton 1987).
20. The text of this *mémoire* is not attached to the minutes. See (Lemonnier 1911–1929).
21. (La Hire 1712, 69): «On remarque ordinairement que lorsque les pieds-droits d'une voûte sont trop foibles pour en soutenir la poussée, la voûte se fend vers le milieu entre son imposts et le milieu de la clef; c'est pourqoui on peut supposer que dans la moitié supérieure du demi-arc, tous les voussoirs sont si bien liés les uns aux autres, qu'ils ne forment que comme une seule pierre: et c'est sur cette supposition et sur la solidité de la fondation où les pieds-droits sont assis, que l'on établi la démonstration de la regle que nous trouverons dans la suite».

22. See the extensive references contained in (Becchi and Foce 2002). Only Jacques Heyman exhibits great caution; see, for example, (Heyman 1972, 82–84 and 168).
23. The *Académie* was to enlarge once again on these two issues on 13th March 1713, in the presence of Bullet and de La Hire, when treating «de la construction et de la poussée des voûtes et aussy de la construction des murs de terrasses». A similar correspondence was to be reiterated in the report drafted by the *Académie des Sciences* on Couplet's *mémoire* referred to above: «Aprés ce que M. Couplet a donné sur les Revêtements des Digues, Chaussées, &c. il étoit naturel qu'il pensât aux Voûtes, dont la Théorie doit dépendre des mêmes principes de Méchanique». See (*Histoire de l'Académie* 1729). It is also well known that C.A. Coulomb was to turn his mind to the same problems.
24. See 17th May 1686.
25. (*Histoire de l'Académie* 1699, 124–127). The *mémoire*, read by Amontons on 20th June 1699, is quoted in (Amontons 1699a).
26. (Amontons 1699a). We owe to Amontons a second, fundamental *mémoire* on the subject, read to the *Académie des Sciences* on 19th December 1699 (Amontons 1699b).

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Conservation of traditional earthen architecture in the Abruzzi region: The construction site for first aid rehabilitation work of Casa d'Arcangelo at Casalincontrada (Chieti, Italy)

Mauro Bertagnin
Gaia Bollini
Gianfranco Conti
Clelia Mungiguerra

Beginning in the seventies Italy has seen an increasing interest in the conservation of traditional earthen architecture heritage. The first Italian forum to promote conservation was the exhibition «Le case di terra: memoria e realtà» (Pescara, 1985).[1] Different events followed that encouraged the study, research, conservation and restoration of traditional earthen architecture heritage, as well as the practice of earthen building techniques, mainly in the field of sustainable building. To support these goals institutions were also established to promulgate earthen architecture heritage. The LATERIS[2] laboratory was set up at the University of Udine in 1983. In 1993, the *Centro di Documentazione Permanente sulle case di terra* of Casalincontrada (Chieti, Italy) was founded. At the Architecture Department of Cagliari University the *Centro Studi e Ricerche sull'Architettura Regionale in Terra Cruda* has been working since 1997.[3] These centres mainly cooperate with the *Rete dei Comuni della Terra Cruda.*[4] This network, officially born in 2000, links those towns characterised by an earthen architecture heritage.

THE CONSERVATION STRATEGIES: TOWARD THE *FIRST AID CONSTRUCTION SITE*

Conservation strategies in Italy are basically related to the restoration of public buildings. The basic aim of these strategies is to improve the awareness of local public administrations as well as the community and local population on the knowledge and conservation of the local earthen architecture legacy.

For what concern the private buildings, urban as well as rural architectures, mainly houses or farms, only few cases of spontaneous restoration were carried out in the Italian regions where the earthen architecture heritage is relevant.[5] In the Abruzzi and Piemonte, some public authorities have just ordered a census of the local rural and urban earthen architectures.[6]

Apart from these sporadic conservation actions carried out by the owners, a large amount of existing earthen rural architecture is generally in an increasing state of decay.

Key reasons for this deterioration include:

- a lack of awareness of the importance of maintaining traditional earthen architecture as a part of the cultural architectonic heritage;
- the general idea, of structural poorness or blighted appearance related to the earthen building, especially private houses, that foments the owners lack of interest in restoration or conservation;
- the frequent belief that earthen architecture restoration costs are onerous and that no artisans or construction firms have the necessary know-how and competence for restoration needs;

— the erroneous idea that earthen rural architecture is insalubrious.

As a result, the owners often ignore the ongoing decay process so that once totally run-down a «new» house can be built. This «new» house is related to the owner's perception of social and economic progress, obtained through the use of «modern» materials such as concrete blocks, steel, etc., which are regarded as an affluent society status symbol.

To oppose this perspective, in the framework of alternative restoration policies, promoted by the local ONG Terrae Onlus,[7] an experiment of *first aid conservation field* was carried out in the case study of Casa D'Arcangelo in Casalincontrada (CH), an old rural earthen house.

To avoid the complete decay of this house a team, directed by Mauro Bertagnin (Department of Civil Engineering-Udine University and CRATerre member), «invented» and promoted a *first aid conservation construction site*.[8]

The basic goals of the field were:

— to promote a *multidisciplinary research* project on the local building technique, the *massone;*[9]
— to obtain the *survival* of the building for the duration of the conservation project;[10]
— to improve local *awareness* of conservation strategies;
— to provide different educational steps to local manpower (such as masons, artisans and designers) to promote the conservation and restoration process of local earthen legacy.

THE FIRST AID CONSTRUCTION SITE AS BASIC CONSERVATION TOOL

The conservation field steps were:

Theoretical training

Theoretical training provided basic notions of technology and typology of earthen architecture heritage in Italy and on the *massone* architecture in the Abruzzi. Furthermore an important part of this section focused on the importance of earth as raw material. Subsequently Mauro Bertagnin and his staff explained and showed the basic earth tests[11] to analyse and recognise the fit earth for the different earthen techniques. Figures 1, 2 and 3.

Figure 1
Preliminary earth tests. Alcock's and lustre test. (Photo Mauro Bertagnin)

Figure 2
Preliminary earth tests. Adhesion test. (Photo Mauro Bertagnin)

Facts about correct static restoration as well as on earthen building maintenance and conservation practice codes were also provided.

An analysis and a survey of external and internal housing deterioration conditions completed the theoretical approach.[12] Figure 4.

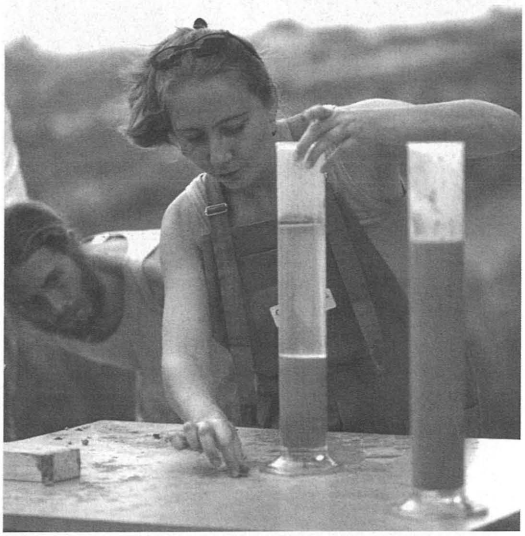

Figure 3
Preliminary earth tests. Sedimentation test. (Photo Mauro Bertagnin)

Figure 4
Damages survey. (Photo Mauro Bertagnin)

Organisation of the construction site[13] and massone manufacture.

The first part of this section was devoted to the explanation of the preparation of a correct and safe construction site.[14] The second and most conspicuous part centred around the practical manufacture of the *massone*. The various *massone* production phases were set in different areas of the field:

— earth sieving and its subsequent crushing by the muller, Figure 5;
— straw cutting into short threads, Figure 6;
— sand preparation for the plaster;
— raw materials mixing (earth, straw and water),[15] Figure 7;
— *massone* manufacture,[16] Figures 8a and 8b;
— *massone* storage in the straw.

Practical work section

According to the problems pointed out during the survey, the preliminary restoration actions, essential to avoid the definitive building decay, were carried out.

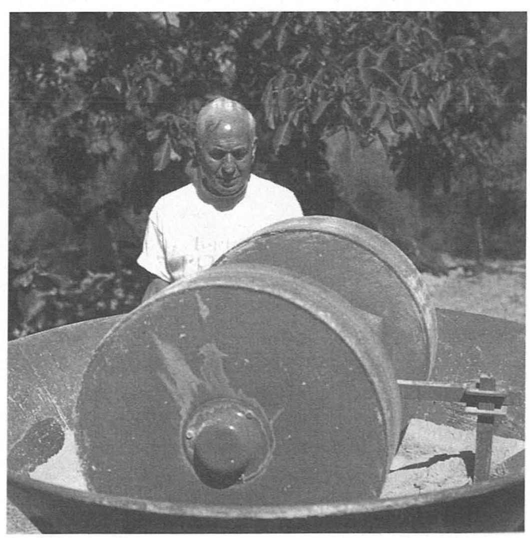

Figure 5
Massone production phases. Earth crushing by muller. (Photo Mauro Bertagnin)

Figure 6
Massone production phases. Straw cutting. (Photo Mauro Bertagnin)

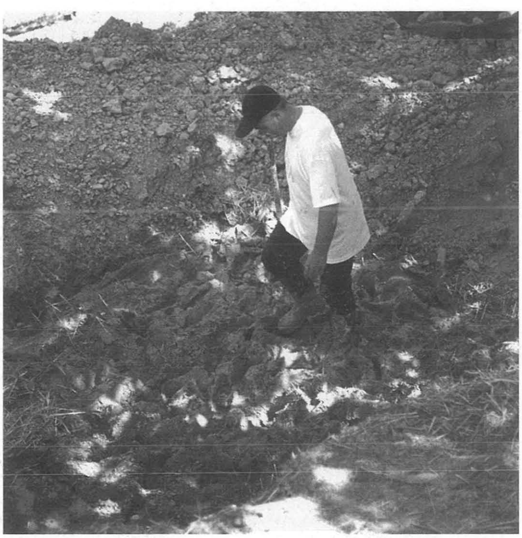

Figure 7
Massone production phases. Raw materials mixing. (Photo Mauro Bertagnin)

Figures 8a and 8b
Massone production phases. *Massone* manufacture. (Photo Mauro Bertagnin)

The team worked basically towards masonry reinforcement, having the building displayed vertical cracks. Those were appropriately stopped using natural elements[17] to re-establish the structural continuity.

The corners too were maintained employing some vegetal elements that contributed to mould and to support the pieces of *massone* gradually added. Figures 9 and 10.

The base flowing water effect was avoided recreating new «good boots» trough covering the base with a fired bricks «skin», Figure 11, once cleaned up its surface and reintegrated the eroded portion. Figure 12.

Figure 9
Corner reconstruction. Corner cleaning. (Photo Mauro Bertagnin)

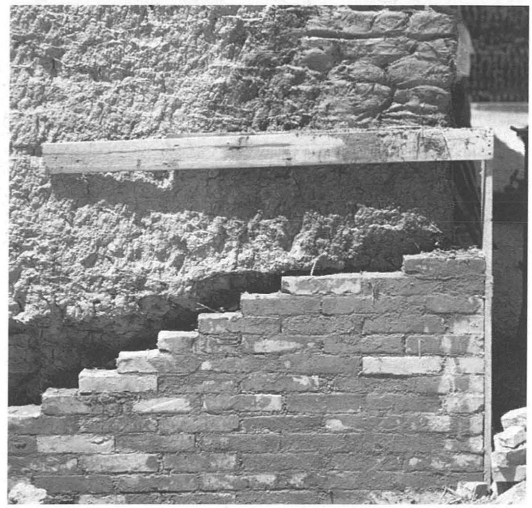

Figure 11
New fired brick basement. (Photo Mauro Bertagnin)

Figure 10
Corner reconstruction. New *massone* insertion. (Photo Mauro Bertagnin)

Figure 12
Final outcome of the corner restoration. (Photo Mauro Bertagnin)

Figure 13
Wall damping. (Photo Mauro Bertagnin)

Before executing all these operations the wall surfaces were dampened, Figure 13, so that the added earthen parts could stick well on the old wall.

Finally some possible finishing solutions were tested on the restored walls. Once identified and defined some testing surfaces on earth-based plasters[18] were used. The sand percentage and its granulometry were varied, and some natural additives added (casein, etc.). Different plaster colours were also tested by adding natural oxides.

To make possible a comparison it was employed an industrial pre-mixed earth-based plaster too.

To satisfy the second basic rule of a good earthen construction («a good heat»), a general overview and check of the tiled roof protection was carried out.

A NEW TOOL FOR EARTHEN RURAL ARCHITECTURE CONSERVATION

This experience has revealed that the *first aid construction site* is an important operating tool in earthen architecture conservation. It allows the contemporary achievement of providing maintenance that can't be postponed, and preparing an appropriate base for further conservation and restoration actions.

This new tool, according to the evaluation made some months later enabled minimal survival of the building, avoiding its sure collapse and helping the subsequent setting up of the restoration project in a longer time. At the same time it has looked as an important theoretical as well as practical educational moment.

Once ended, the *first aid conservation construction site activities* made it possible to think to a more meditated restoration project. This final project will be in fact set up according to the typological technological, philological and historical investigations concerning the earthen building. To provide a correct conservation strategy the integration in the local environment will be considered too.

NOTES

1. See Morandi, M. and F. Profico, (1985).
2. LATERIS laboratory is involved in activities and researches on earthen architecture, sustainable building, recycling in construction practice, alternative and renewable sources of energy, as well as on the rediscovery and revaluation of the traditional building techniques and know-how.

3. In the '80-'90 decade some others laboratories have been brought in operation in different Italian universities. For an exact survey see Bollini (2002).
4. It's a national network that aims to coordinate the municipal administrations actions on the matter of earthen architecture.
5. Sardinia, Piemonte, Marche and Abruzzi are rich of earthen rural and urban architecture examples.
6. All the related expenses have been adopted in the framework of Regional Found and the census results were often published. For an example see Conti (1999).
7. Terrae Onlus is an association of earthen architecture connoisseurs, which locally promotes events focused on the earthen architecture heritage. It mainly operates in the Abruzzi region. Terrae Onlus often works in collaboration with Udine, Cagliari and Pescara universities.
8. This unique experimental construction site for first aid rehabilitation work was carried out with the collaboration of Maria Cristina Forlani (Faculty of Architecture-DiTAC-Chieti University), the ONG association Terrae Onlus (Chieti, Italy), the Scuola Edile di Chieti and thanks to Mr. D'Arcangelo, the owner of the earthen house. About twelve persons, besides the organisational staff, have participated; it was a very heterogeneous group, consisting of several local architects, an historian, a geologist, some masons, some final year students of different disciplines and people interesting to the oneself-building traditional and local techniques.
9. The international name of the *massone* technique is cob. To have further information on cob you can see Houben and Guillaud (1994): 178–179. For the *massone* technique and its diffusion in Italy see Bertagnin (1999): 183–219.
10. The conservation project is obviously related to the financing process that can be carried out by the owner following an affordable timetable.
11. All the explained tests were carried out following the CRATerre Standard for the earth tests. See Houben and Guillaud (1994): 131–144.
12. This final section was carried out under the guide of the architects Stefania Giardinelli e Cinzia D'Arcangelo (members of the Terrae Onlus staff).
13. The whole logistical framework of the construction site was set up by the architect Gianfranco Conti, president of the ONG Terrae Onlus, and his staff.
14. A master mason of the Scuola Edile di Chieti explained how to prepare a building safety standards respectful field.
15. This operation was carried out in a hole where the earth, the straw and the water were mixed by feet. Traditionally also cows and draught animals provided mixing «power».
16. It consists in shaping by hand a cake of earth and straw. The participants divided into groups supervised and co-ordinated by the architects Stefania Giardinelli, Cinzia D'Arcangelo (Terrae Onlus), Raffaella Petruzzelli (Chieti University-DiTAC) and Gaia Bollini (Udine University-Department of Civil Engineering) experienced in turn all the different stages of the massone production.
17. The natural elements were sticks of bamboo or straw and small wet *massoni.* In this phase these natural elements were used as seam and connection elements between crack borders.
18. These plasters have been manufactured in the construction site.

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Foundations and wall structures in the basement of the Colosseum in Rome

Heinz Jürgen Beste

Following a generous offer made by Rome's Archeological Superintendence, the German Archeological Institute in Rome commenced an investigation of the construction details of the arena and the basement of the Colosseum in 1996. The Colosseum forms part of a series of well preserved major structures of antiquity whose state of preservation is inversely proportional to our knowledge of the history of the building.[1]

The ultimate aim of the work we have commenced is to distinguish the multitude of building phases that followed each other in the five centuries in which the world's largest amphitheatre was in use.[2] Moreover, it is proposed to attempt design reconstruction based for the first time on a thorough cataloguing of building details and history. The Superintendence also expect a proposal for using the basement as a museum, as also for the restoration of the arena floor.

The basement of the amphitheatre, which several walls subdivide into a series of corridors, measures 76.12 m along the longitudinal axis of the oval and about 44.07 m along the shorter transverse axis. It is delimited by the so-called encircling wall, which in its day carried the podium. Constructively this wall is the counterpart of the so-called foundation wall which supports the outer facade. If one today approaches the edge of the arena from the stands, one looks down at the basement, because the wooden arena floor has long since disappeared. The longitudinal axis of the oval-shaped basement is oriented in the East-West direction, and this is also true for the greater part of the walls one finds there. These walls, made of tufa blocks and bricks, subdivide the basement into twelve corridors of different widths and lengths and, more precisely: six elliptical corridors that run parallel to the encircling wall and nine corridors parallel to the longitudinal axis (fig. 1).

Phase I

At the beginning of our investigations attention was concentrated on distinguishing the individual building phases and, more particularly, the tufa walls, because these divide the basement into the aforementioned corridors. For this reason I shall henceforth describe these tufa walls as Phase 1. As reconstructed on the basis of our building records, this phase is characterized by a very light but long-spanning construction. The walls, which are about 90 cm wide and 6.30 m high, attain arch spans of up to 4.0 m. These filigree walls made of tufa, which are set between 2.0 and 4.0 m apart and carried the wooden arena floor, were presumably at first underdesigned for withstanding the vibrations caused by earthquakes, ground subsidences and perhaps the games themselves and therefore had to be subsequently stabilized, as is clearly brought out by our findings (fig. 2).

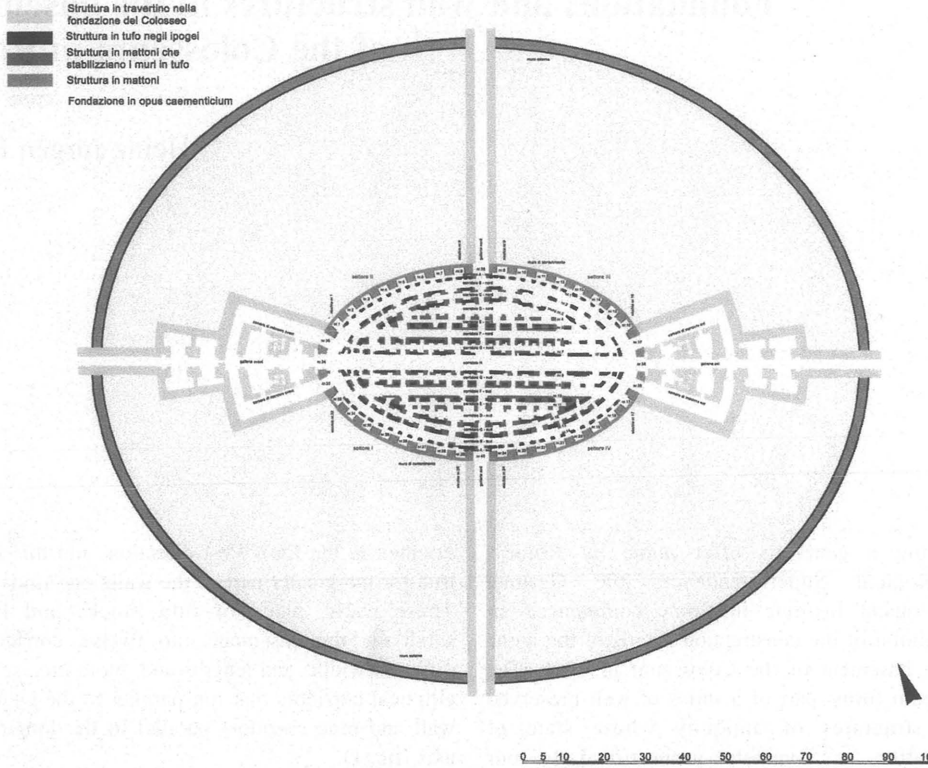

Figure 1
Foundation platform and basement of Colosseum, general site plan

Phase II

For this reason all the openings in the arches were reduced by inserting brick arches 60 cm deep within the existing ones. What is particularly noteworthy about this work is that the soffit was covered with *sesquipedales* and *bipedales*, thus making it possible to use falsework instead of complete centering. At the same time as the arches were consolidated, all the openings and passages were narrowed by providing each soffit with a strong brick pillar (*c*.60 × 60 cm). The only reason why we assume that the walls numbered 4 and 11 belong to phase II is that they are made out of brickwork with a basic tufa layer. I suppose therefore that in phase I there may have been wooden piles instead of the walls 4 and 11. There are numerous square structures in the basic tufa layer which I believe functioned as the pile holes for this wall. For reasons of construction the walls 4 and 11 could not have been built at a later stage as they support walls belonging to phase III.

Phase III

This phase consisted of stabilization involving all the walls. It repeated the procedure of phase II by adding masonry in all the passages, which were thus further reduced in size. The tufa walls in corridors C, E north, E south, G north and G south were each reinforced by the addition of brick walls rising to a height of *c*.3 m; they were joined to each other by means of transverse arches and thus gave rigidity to the original walls (fig. 3).

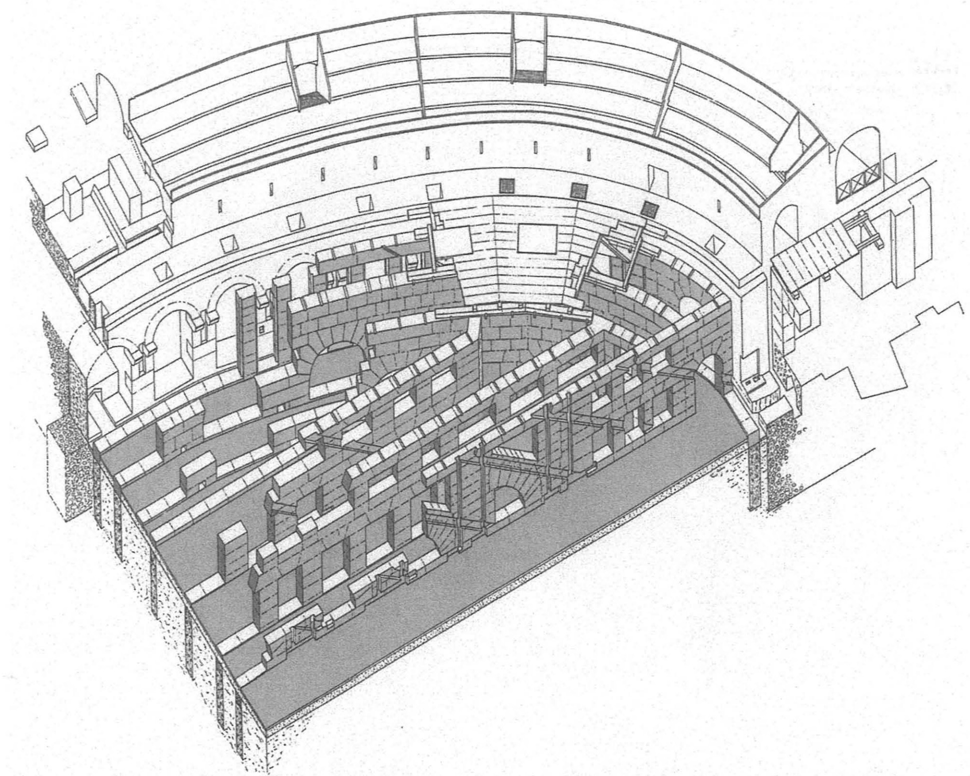

Figure 2
Reconstructed scheme of the basement, Phase I

Phase IV

Phase IV constituted a very substantial alteration of the existing subdivision of the basement because it involved the erection of a brick wall (*c.*2.3 m high) in corridors F north and F south that closed off the greater part of the passages that served them. This phase is distinguished from all the others by the fact that it does not consist of bricks of uniform size and rests on a pediment made of material that must have fallen down from the top of the walls.

Theoretical treatises about technical details or traditions regarding the design and dimensioning of foundation are not to be found in the literature of antiquity. Vitruvius, who provides us with quite a few constructional details, dedicates a brief subchapter to the theme, but never really comes to grips with the problem.[3]

The state of research of modern times reflects the lack of interest of the classical authors. Every description of the building speaks of the foundation and the manuals do of course provide an summary overview of the various solutions, but fight shy of a true analysis of the problem.[4] Though the foundations are literally fundamental for every building structure, only a few papers have hitherto been dedicated to this theme.[5] And this notwithstanding the fact that some classical authors speak of building failures during construction due to inadequate foundations.[6]

The fact that the foundation is the sole part of a building that cannot be designed by means of permanent correcting trials during construction shows us that the problem must be considered to have been solved in the case of the monuments that are still standing today. Although considerable interest has always been shown in the structural concept and the

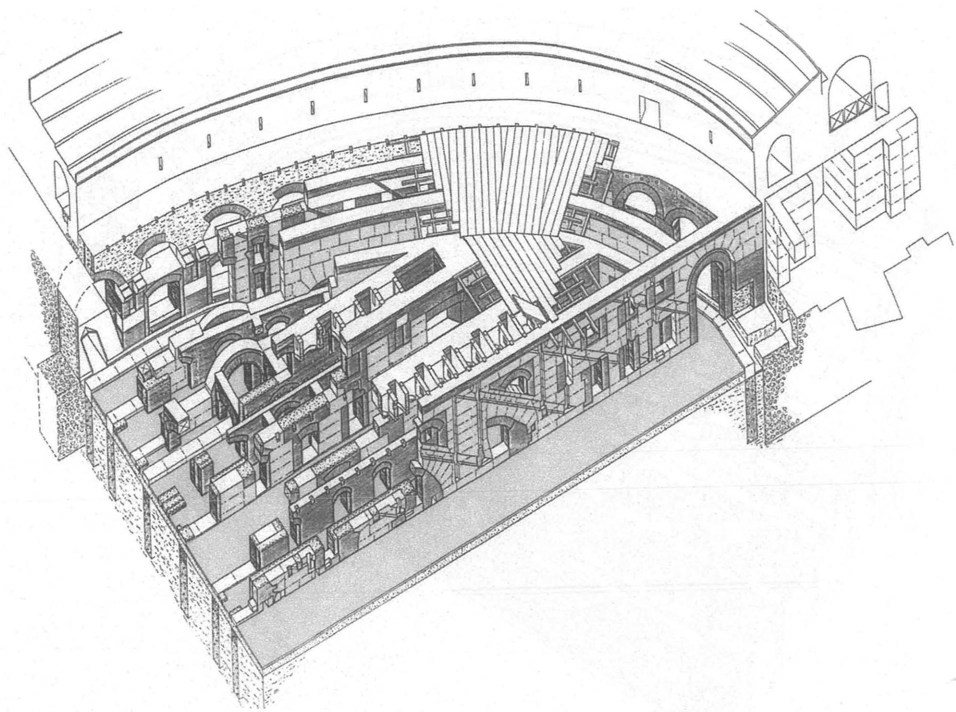

Figure 3
Reconstructed scheme of the basement, Phase III

static safety of the Colosseum, the state of our knowledge about the foundations of the building is very scant.[7] Presumably the surviving ancient structures and especially the Colosseum suggest a stability and durability such as to make us tacitly assume that they are adequately founded.

In his book entitled *Ingegneria romana(1927)*, Giuseppe Cozza was the first author to consider the problem of the foundation of the Colosseum. According to his concept, which he developed in the light of the structure of the eastern gallery (38), the 80 walls that carry the roughly 50 m high superstructure and cavea of the Colosseum do not stand on a foundation platform, but rather on strip foundations made of *opus quadratum*, each roughly 3 m wide and reaching down to a depth of about 6 m. In order to assure a solid support and thus enhance their carrying capacity, these eighty strip foundations —in the opinion of the author— are joined by transverse arches, while the space between them has been backfilled with soil.

For a long time the theory formulated by Cozza about the structure of the Colosseum's foundations was neither doubted nor checked, and it was only in 1977 that a trial boring executed in the area of the outer circumference showed that the foundation consisted of *opus caementicium*, and not *opus quadratum* as had previously been assumed.[8]

The reconstruction of the arena floor commenced in the year 2000 brought with it the possibility of sinking six sampling cores in the basement to clarify the foundations on which it rested. Three of these were situated in walls 7, 8 and 13 (S1–3) and a fourth in corridor H (SA). A further trial boring was carried out in niche 39 (SB) and yet another (SC) between niches 15 and 16. The latter, however, was horizontal rather than vertical (fig. 4).

The results of borings SB and SC, considered

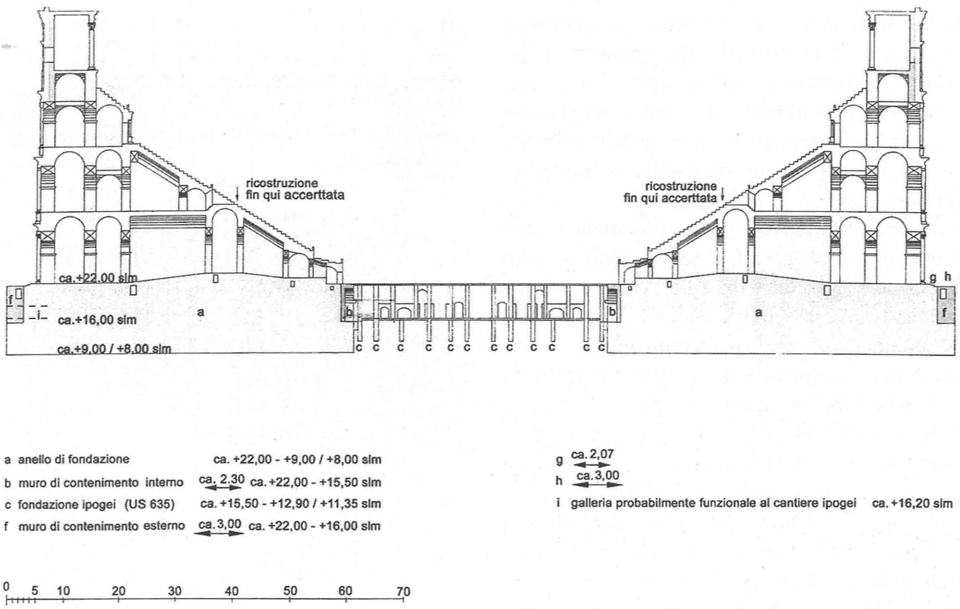

Figure 4
Section through the shorter transverse axis from the Colosseum

together with those of the 1977 trial, made it possible for the first time to make some concrete statements about the construction process and the size of the foundations. Accordingly, the walls in the basement would seem to have a separate foundation, while the façade and the cavea of the Colosseum rest on a foundation ring made of *opus caementicium* that is about 60,30 m wide and 12–13 m deep. The manner in which it was constructed permits its being subdivided into two sections. The coring results show that the shallow footing of the ring is situated at a level of 10.0 m asl. Section I of the foundation rises from this level and reaches up to about 16.0 m asl. It is now documented that it was constructed with the help of timber shuttering, which actually be seen in the basement at a distance of about 0.50 m below the upper edge of this first section. Section II of the foundation reaches from about 16.0 m asl to about 22.0 m asl and is lined both on the side of the façade and on the inside (i.e. in the area of the basement) by a 2.30 m wide brick wall, the so-called inner and outer encircling walls.[9] From the construction point of view, the fact that different techniques were employed for the two sections means that the lower section must have been built below ground level and with the help of formwork.

To permit the construction of the two encircling walls, on the other hand, the surrounding ground must either have been excavated down to level 16.0 m asl or must not have risen above that level in the area in the area in which the Colosseum was erected, our state of knowledge suggesting the latter assumption to be more probable. This assumption is also confirmed by ongoing excavations, which show that the level of the buildings destroyed by the fire of 64 A.D. is of the order of 15.50 m asl and therefore lies only some 0.50 m above the floor level of the Colosseum basement.

The result of the horizontal boring SC through the inner encircling wall made it clear once more that the brick wall, quite apart from its architectural articulation into 40 niches, performs first and foremost a structural function. The wall sections between the niches, 1.80 m wide by 2.30 m deep, act

as buttresses or counterforts and therefore enable the brick wall, which within the niches has a thickness of no more than 0.72 m, to resist the pressure of the foundation ring between 16.0 m and 22.0 m asl. Whether the outer encircling wall is constructed in the same manner has to remain an open question for the moment, because so far our knowledge is limited to its uppermost layer.

Trial borings S1 and S3 in the basement of the Colosseum showed that walls 7 and 8 stand on strip foundations that are dug into the ground of the basement and have a width of about 3.25 m. The material contents of the two cores enable us to distinguish two sections of these strip foundations. Section I, which has a depth of about 3.25 m and extends between 12.75 and 15.50 m asl, is made of *opus caementicium* and is topped by a layer having a thickness of the order of 25–50 cm, the so-called levelling course (16.0 m asl), likewise made of *opus caementicium*. The contents of core S3, which reached down to 11.20 m asl for an overall length of 4.80 m, consists exclusively of *opus caementicium*, but in this case the levelling course cannot be demonstrated. The results of the two borings are very clear and show that the 6.0 m high walls in the basement rest on foundations having a depth that varies between 3.25 and 4.80 m and are therefore adequately founded. The borehole in corridor H (SA) showed that only the levelling course was poured there and that it has a thickness of about 50 cm (fig. 5).

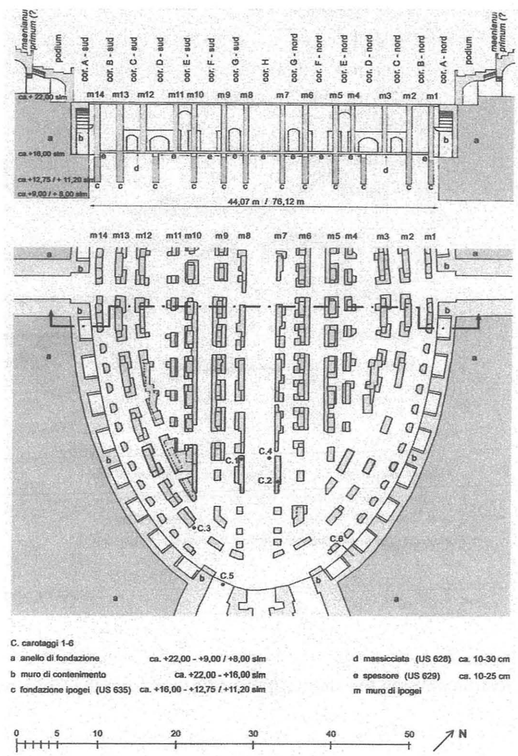

Figure 5
Detail plan and section through the shorter transverse axis of the basement

The archaeological investigations undertaken at the same time made it clear that the structure of the foundations in the basement is considerably more complex than is suggested by the results of the coring. Four of these (S1, S6, S19 and S 27) are of particular interest: they brought to light wooden residues along the strip foundations and thus made it possible to conclude that these must have been constructed within 4–5 cm thick timber formwork, which —given the high water table in this area, has been well preserved to this day.

Though the high water level prevented the boreholes from reaching a really substantial depth, they showed that the strip foundations of wall 1, 5, 6 and 14 consist of three sections, which contrasts with those of wall 7 and 8, which consist of only two sections. The layer of *opus caementicium* (US 635) is followed by another layer of *opus caementicium*, though in this case having a different composition. The upper layer has the same width as the lower one, but its height varies from one wall to another. Just as in the case of walls 7 and 8, it is topped by a levelling course (US 629). Having a thickness of the order of 10–15 cm. It is not clear why the structure of the foundations should vary from one wall to another. All I can do is to suggest that the subdivision into several sections might be due to unevenness of the terrain in the basement area. Presumably the lower section of *opus caementicium* was in each case at first brought up only to the level of the adjacent ground. The levelling was then obtained by means of the second layer of *opus caementicium* poured on top of the first, so that the height of the second layer must have depended on the ground level. The levelling course, which covers the entire area, i.e. both the corridors and the strip

foundations, then brought the structure up to 16.0 m asl, the desired floor level for the basement. Since the foundation section made of *opus caementicium* of walls 2 and 13 neither has a so-called cement-gravel bedding nor is topped by a levelling course, it seems possible that these two foundations might have served as level marks. They presumably also served to define the location and direction of the other foundations, since these —following the pouring of the levelling course— were no longer exposed to view.

Conclusions

The roughly 50 m high superstructure/façade and the cavea of the Colosseum rest on a foundation ring consisting of some 246,000 m^3 of *opus caementicium*. The enormous mass of the foundation seems to be adequately dimensioned, because parts of the southern side of the structure collapsed only in the 12 century, presumably as a result of strong earthquakes. The fact that the thesis according to which the Colosseum rested on strip foundations of *opus quadratum*, though formulated in 1927 without a detailed investigation, had to wait 50 years before it was corrected is indicative of the limited interest that is still being dedicated to structural problems, and this notwithstanding the fact that they are altogether elementary for the building in question.

Furthermore, recent data about the structure of the foundations and knowledge of the street level of the buildings that previously stood on the site of the Colosseum enable us to make the following hypothetical statement about the Colosseum area. Unlike the western side of the valley, it would seem that this area was not redeveloped after the fire, because otherwise the second section of the foundations with the two encircling walls could not have been constructed in this particular manner. That the area kept clear of the housing reconstruction undertaken by Nero, which reached levels up to about 20.0 m asl, might have been constituted by the *stagnum Neronis* seems plausible from a constructional point of view and would partially explain the short time it took to put up the Colosseum, because no excavation work would have had to be undertaken from this level down to 16.0 m asl. However, our knowledge of the level relationships between pre-Neronian buildings and Nero's reconstruction in the area around the Colosseum is as yet extremely scant, so that a more precise concept of the size and location of the *stagnum* will have to await the drilling of further trial holes.

Notes

1. For the history of the Colosseum see Parker 1876; Rea 1996; La Regina 2001. For the gladiatorial combat e games see Golvin and Landes 1990. For the amphitheatres see Golvin 1988; Wilson Jones 1993.
2. On the individual phases and the function of the various corridors in the basement, see Beste 1998, 106 ff.; 1999, 249 ff.; 2000, 79 ff.; 2002, forthcoming; Beste and Schingo 1998; Rea and Beste 2000, 311 ff.
3. Vitruvius III 4,2.
4. Adam 1994, 115 ff.; Giuliani 1990, 119 ff.; Ginouvès 1992, 7 ff.; Durm 1909.
5. Kienast 1991, 123 ff.
6. Tacitus, Ann., IV, 62–63; Plinius, Epist. 10, 48.
7. Cozzo1927, 204 ff., fig. 133; idem, 1977, 20 ff., fig. 12; Lamprecht 1987, 155 ff., fig. 150.
8. Mocchegiani Carpano 1977, 10 ff., figs. 2, 6; idem, 1985, 122 ff., fig. 1.
9. Schingo and Rea 1993, 65 ff.

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Archive documents and building organisation. An example from the modern age

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Anna Decri

Written sources offer abundant material to those who wish to study the history of building. One of the possible fields of enquiry is the building site of the past, in various respects.

In general the data to be derived from archive sources can regard technical aspects, organisational aspects and financial aspects. Often it is the document itself or even the datum itself that can provide different types of information, depending on how it is read and processed.

In the paper that follows we have endeavoured to illustrate the information potentialities of archive documents for the study of the preindustrial building site, with the aid of examples referring to the city of Genoa in the modern age (16^{th}–18^{th} centuries).[1]

Many types of documents contain useful information. Considering just a few of them we can mention building contracts and specifications with relevant project drawings, contracts referring to production, sale and purchase of building materials, testimonies, judgements or court deeds regarding controversies in the building sector, receipts of payments of labourers and materials, estimates, inventories, account books, laws on building, guild statutes, treatises and manuals, deeds of the building magistrature, and so forth.

The different types of document may be more or less useful, depending on the specific objective of the research. For example, it is one thing to seek sources for the history of a specific site and another thing to seek them for the history of a site in a given epoch and a specific territory.

If the objective of the research is the history of a specific territorial and temporal context, one important aspect of the study of the written source is that the document must not constitute an isolated datum but must be part of a series of documents that is sufficiently complete in time and/or space, i.e. such as to permit interpretations based on fairly large numerical bases.

If data are collected in a systematic way from a large number of documents referring to a homogeneous territory, it is possible to reconstruct the building site in its most widespread modalities and also to identify exceptional cases and moments of innovation.

Through systematic comparison of a large number of documents, one can also study the technical language used, generally characterised by local terms which are often archaic and difficult to interpret. Without a real understanding of the technical language, one risks misunderstanding the real content of documents or neglecting significant parts of them.

Realisation times

Notarial contracts always set the time for execution of work: in the sample examined it varies from a minimum of 15 days to a maximum of nine and a half

months, with an isolated case of 18 months allowed for the delivery of a big supply of marble elements.[2] For example, to add another floor onto a modest house,[3] a deadline of two months was set: the work included scaffolding, execution of the walls, putting in windows, plastering, execution of the roof surface and chimneys. Two months were also contemplated for various jobs connected with reorganising and restructuring a nobleman's residence:[4] closing off the existing loggia by building an atrium covered by a vault, redoing and widening the stairs, execution of a column arcade on two sides of the cortile, alteration of the windows, putting in new door jambs, plastering and whitewashing of various parts of the house, execution of some floorings and various finishing touches. Lastly, reconstruction of a building of at least three floors above ground, including flooring and plastering, was to be completed within seven months.[5]

These data reveal marked celerity in the execution of work, if the times contemplated really were respected. A confirmation of this —for work, incidentally, that was very big— comes from documentation on work done at the harbour. In the case of Ponte Calvi, one of the disembarkation bridges reconstructed in masonry in the 15th century, the work was completed in all details just over five months after the stipulation of the first contracts (Boato et al. 1993, 76).

Preparation of the building site: excavation, flattening, demolition

The historical city of Genoa developed on a terrain affected by more or less steep hills and valleys with short watercourses running through them. The site of any construction generally sloped to a greater or lesser extent and was affected by the presence of a rocky subsoil (loamy chalks or marls). Hence setting up a building site frequently meant reckoning with the need to perform excavation or flattening of the terrain, and sometimes also with work for redirecting and channelling water. In the 16th century, after the flourishing development of the late Middle Ages, the city within the walls was complete. Hence often the building site often had to come to terms with the presence of older buildings (intact or in a state of ruin), to be demolished or incorporated in the new construction.

Diggings, whether linked to partial land excavation or the realisation of foundations, were entrusted, either directly or by subcontracting, to workers able both to carry out the operations and to take away the material that could not be used or disposed of *in loco*. For this reason we often find contracts given to muleteers.

The costs of digging operations depended on whether the subsoil was made up of compact rock (*scoglio*), marl (*tovio*), rubble (*zetto*) or earth. In the case of mixed situations, every type of digging was evaluated differently. This was related not only to the different effort or difficulty, but also to the possibility of recovering material that might in some way be useful for construction work.

The most precise contracts were full of clauses contemplating all eventualities, not only so as to establish competencies and costs, but also in order to define the execution modalities of every possible type of work. In a contract dating from 1623, for example, it is established that if during the course of the excavation fragments of stone and pieces of brick (so-called *frazzi*) are found that are judged to be still suited for use, these will have to be set aside. However, this will not affect the cost of the operation. If, instead, larger stones are found (defined as stones *da canella*), these will have to be deducted from the volume of the excavation and computed separately.[6]

Earth and rubble, when possible, are used in the construction work itself or disposed of on the property of the client: earth alone and earth mixed with rubble for flattening of the land under or around the building, rubble for filling vaults or for putting under flooring.[7] In cases in which the terrain was of a clayey type (this were rare, seeing the nature of the Genoese soil), the raw material for making bricks was recovered.[8]

In general one notices an attitude of great attention serving to minimise the quantity of material to be taken away. The fact is that in the city there was a problem linked to the risk of silting up the harbour, and a contribution could be made to this both by material dumped in the sea and material deposited in all the area behind the harbour. Hence it was the magistrature responsible for watching over building activity that decided each time what places could be used for disposing of material and established at what distance from the coast and in what stretches of sea rubble could be deposited.

This attention is also reflected in behaviour towards existing buildings. Even when the project contemplated more or less total transformation of the building, efforts were made to reuse everything possible, minimising demolitions and recovering all possible material. Evidently the organisation of a building site had to take all this into account, both at the time of dismounting various parts and in the selection and management of material resulting from demolition. The latter material might have a different destiny, and this is an aspect on which contractual clauses generally dwell, above all to clarify the financial points: recovery and reemployment on the building site, recovery for future reuse, recovery for uses other than building ones (wood for burning), transfer to public tips.

SPACE MANAGEMENT

Measurement and storage of materials

Supply on building sites, according to the usual custom, was done on a daily basis (*dietim, ad iornatam*), depending on how the work was proceeding. If suppliers respected their contractual obligations, this made it possible always to have the necessary materials available, without having to allocate big areas for storing them. However, a space always had to be set aside for unloading material, so as to undertake the necessary measurement required for checking the quantities delivered, and so as to have available what was necessary for the work being done.

Timber and iron were stored inside, to protect them perhaps against humidity or perhaps against thefts: this at least is what happened in the work for the New Walls (Bruzzo 1935, 24).

Shapeless stones of medium-small size (so-called stones *da canella*) had to be carefully piled up, to permit a first estimate, on the ground, of the quantities supplied. This operation was called *acanellare*.[9] The measurement and payment of these was in accordance with the volume (the so-called *canella da muri*, equal to 288 cubic *palmi*, amounting to 4.4 m^3).

In a detailed contract relating to the «Albergo dei Poveri», indications are also given as regards the movement of materials. The building contractor has not only to have stone material deposited on the site, in accordance with the common practice, but is bound to deliver it to the masons in the places where they are working. Rough-hewn or squared-off stones (so-called *piccate*), characterised by high weight and size, are to be heaped up at the foot of the pilasters that are erected with them and then raised by means of a crane as the work proceeds. By contrast, small stones (so-called stones *da canella*) are to be taken at once onto the scaffolding, to be handy for the masons.[10]

Lime, carried on a mule's back, had to be unloaded and weighed, in order to register the quantities of quicklime effectively delivered. Almost all contracts specify that from the supply agreed on there are to be deducted the *crudi* (stone parts that are not sufficiently calcinated, unsuited to the production of slaked lime), with the implicit admission that a lime which is *buona e mercantile* (i.e. good and suitable for commerce) like that requested might contain some. It is likely that the presence of *crudi* only became evident after slaking and some form of filtering.[11] Only then could the *crudi* be weighed, so as to deduct them from the final calculation. Then the waste material had to be collected, loaded on mules and taken away from the site.[12]

Water supplies

An indispensable material on the building site is water. Necessary in big quantities for slaking lime, water was also used in many other operations: wetting bricks before putting them in place, wetting plaster to make sure it took hold, tempering iron tools . . .

In some cases water was collected in cisterns which would then serve for later site uses, and in other cases taken from an aqueduct or from wells in the vicinity to the site in the quantities necessary for the various jobs: when the organisation of the site allowed it, appropriate channels could be put in place, and otherwise it was transported by hand or by animals.[13] One can suppose at all events that there was a small supply kept in barrels or something similar, from which workmen could take water for daily use.

Lime pits

There is quite a lot of proof that on the preindustrial site lime slaking was done on the spot. The Genoa documents too clarify the fact that lime came to the

site in the form of lumps (*motti*), and that labourers, including women, proceeded to bathe the lumps to transform them into slaked lime.[14] Hence it was necessary to allocate a space, inside or outside the construction, for this operation.

Current building practice establishes that it is useful to have a container (*bagnolo*) in which to perform slaking and a pit dug out in the earth in which to pour the slaked lime thus produced, after filtering, to season it. The whole operation could be performed in a single tub, as we are informed by Francesco Milizia and as historical pictures show.

It is a common opinion among authors ancient and modern that, in order to obtain a good product, seasoning must go on for several months, if not years. We have no notices regarding whether in Genoa building practice such seasoning was performed, but it does not seem it could have been very long, seeing that lime supplies were brought every day in accordance with the site's needs (*ad iornatam, daily according to needs*). The presence of several lime pits on one site, in addition to allowing the preparation of a larger quantity of material, permitted a rotational use of pits and hence maceration of the material contained in them. However, in the historic area and in small restructuring jobs or maintenance of existing buildings it is possible that people had to adapt to the small spaces available. Once the work was done, evidently the pits had to be filled in again.[15]

Preparation of materials

Work was done on the site that often required dedicated spaces: mixing mortars, washing stones, making metal objects, cutting wood and marble . . . Such spaces could be created inside the building or in adjacent free areas. In this case too the type of work and the size of the construction were of some importance: on restructuring sites a covered space could easily be found on the ground floor of the building; in work like the New Walls or the Carignano basilica shacks or shelters were made on purpose.[16]

Lodgings for workmen

When the building site was in a place that was distant from the town or village or involved a large number of workmen, also making it necessary to recruit workers from outside, it might be indispensable to set up some lodgings for the workmen themselves. There are testimonies referring to a situation of this kind in the case of the site for constructing the New Walls (1630–32), which at certain times involved over 5000 workers. On that occasion the public administration erected the shacks for the workers to sleep in and for the so-called *bicazze* (canteens, sales points run by the contractors).[17]

PROVISIONAL WORKS

Scaffolding

Among the clauses in building specifications we also find ones referring to provisional works. The onus was generally on the craftsman, who had to procure the materials necessary for erecting the scaffolding.

A legal regulation of 1594, aiming to protect the safety of passers-by and to avoid damage to the harbour, prescribes making the scaffolding «very safe and shored up with thick planks and *fassine* both under and around» and gives permission to let it lean against neighbouring houses.[18] The *fassine* (fascines of organic material: canes, small wood or anything similar) certainly had the function of preventing the fall of objects (tools, materials . . .) and of small rubble or mortar used in construction. This explains the task of protecting the harbour they were entrusted with: the biggest danger for the harbour was the constant risk of silting, to which there contributed all the dissolved materials that the rains could drag along the roads as far as the sea.[19]

We also get indications on the way of erecting scaffolding from pictures.[20] For Genoa there is a famous scene of the construction of the Trebisonda warehouse, Figure 1, painted by Luca Cambiaso in Palazzo Lercari-Parodi.[21] In it we see scaffolding made up of a frame of wood stanchions and crossbeams laid on the ground. We do not see the protective boards mentioned before, but this can be attributed to an artistic need to avoid masking of the very scene to be represented.

In addition to this type of scaffolding, which is comparable to modern scaffolding, there are notices of a type that could be defined as mobile, Figure 2, in

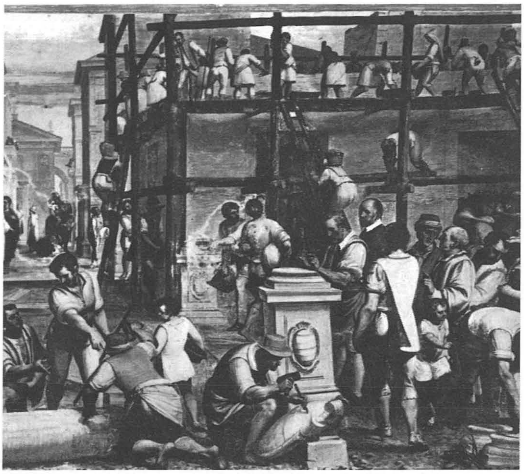

Figure 1
The construction of the Trebisonda warehouse, painted by Luca Cambiaso in Palazzo Lercari-Parodi, Genova

that it consisted of shelves corbelled on the masonry, to be moved as the building advanced.

To get to the different levels of the building ladders were used, but also, when possible, ramps, as it was easier to carry loads up these.

Figure 2
Giovanni Carlone, villa Spinola di San Pietro, Genova Sampierdarena

The materials used were wood for the bearing structure and the gangways and rope for the couplings.[22] These materials, in particular timber, might be rented for the duration of the work or might be recovered from demolitions.

Supporting structures

Also as regards provisional work of another type, supporting structures (*ceitri*) for the construction of arches and vaults, notarial documents do not give technical details. The only information refers to the use of planks and the existence of a specific type of nails: so-called *chiodi da seitro*.[23]

However, an interesting document exists that refers to the construction of the Carignano basilica (1549 on), done by Galeazzo Alessi. With the aid of a written report and two sketches, Figure 3, Alessi compares two types of timbering, defined as *forma ordinaria* (ordinary form) and *forma straordinaria* (extraordinary form).[24]

The *forma ordinaria* consisted in scaffolding that, by means of a central stanchion, was raised from the floor level to the vault level, where, by means of a tree shape, it supported the surface of the planking on which the vault was to rest. According to Alessi, in addition to being inconvenient because of the space it took up on the ground, it would have cost twice as much as the other and would also have been weaker.

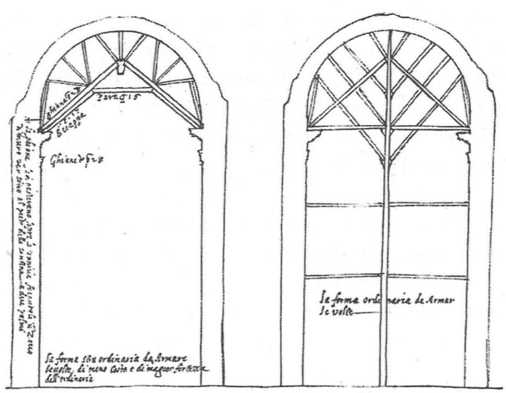

Figure 3
Galeazzo Alessi, Sketches for Carignano basilica, in Archivio Sauli, 112, 30–11–1560

The timbering to be used, resting on the impost cornices, was constituted by a sort of truss that, by means of a reticule of little beams, supported the necessary support surface.

Of particular interest are the motivations of the comparison: in Genoa the so-called *extraordinary* supporting structure was unknown or little used. It is for this reason, and perhaps to ward off any dissents, that Alessi insists on explaining its advantages and the ease involved in making it. One easily realises the possible influence of a «stranger» in the development of new techniques, as well as in the adoption of new forms. However, while the formal aspect can be analysed afterwards, the use of particular expedients does not always leave traces, and only explicatory documents like the one mentioned can preserve the memory of them.

Equipment for raising and movement

The richest source for knowledge of the «machines» used on building sites is the pictorial one. Some indications on instruments effectively used in Genoa can be found in written documents too.

Reference is often made to the use of pulleys (*taglie*) and winches (*argij*) with the relevant ropes for raising beams, blocks of stone and any other material which is very heavy and cumbersome.[25]

According to the terms of the contract relating to the «Albergo dei Poveri», the building contractor is to procure everything necessary for constructing the so-called *calandroni* and the trestles (*cravie*) to be used for raising blocks of stone.[26] However, putting the trestles up is a task for the masons. A similar division of the work is found in other contracts.

There are also some interesting indications on lifting machines in relation to the work done at the harbour. When in the 15th century it became necessary to prolong the Old Wharf, Anastasio Alessandro, *architectus et magister diversorum operum*, was called in, precisely in his capacity as an expert on site fitting.[27] On that occasion *ruote* («wheels») were set up for lifting big stones, both on land and on the boats used for transporting them (*pontoni*).[28] In the *wheel*, which was hollow, there were one or more men, who set it moving with their own weight, thus succeeding in producing greater force than can be obtained with the winch alone. These *gru a ruota calcatoria* or *gru a gabbia di scoiattolo*, known since the classical epoch, (Gille [1978] 1985; Adam 1984) were normally used in the building sector too, as we can see in various medieval and post-medieval illustrations, and there is nothing to stop us from thinking that they were also used on Genoa building sites.

Workers

On a site of some size in the busiest period there might be hundreds of workmen, each with a precise task and hierarchically divided into workmen with various specialisation[29] and labourers. The latter were not yet skilled workers, and some would never be unless they showed the necessary skill; the client might ask for a limited number of some labourers to be involved, to ensure work of better quality.[30]

The managing role was played by the *capo d'opera* —sometimes helped by a *sotto capo d'opera*— who organised the work and the supplies, co-ordinated the workmen and spoke with the client of his representatives (inspectors, fabric deputies, accountants).[31] In his work the financial aspect was not secondary to the technical one: it was he who took on himself the onus of the supplies in the building contracts for which he was paid a lump sum, sometimes even taking on the role of being a supplier of building materials.

On important sites, where it was the clients who dealt with supplies, the *capo d'opera* presented the list of the days worked for people to be paid, and a bond of trust might live on with the clients even after the completion of the building, both for maintenance and for subsequent work.

The qualification of *capo d'opera* —traditionally taken on each time by those designers who, becoming the building contractors appointed to carry out the work, dealt with managing works— was regulated within the «Guild of the Master Masons» (referred to in Genoa as *Antelami*) and thus became a qualification proper, obtained by means of an examination (Boato and Decri 1995, 26).

A figure belonging to the same guild, though in separate lists from the 17th century on (Boato and Decri 1995,), was the *scalpellino* or stonemason, who worked the stone. His role on the site took on different nuances depending on his specialisation: the

quarrier and supplier of stone material, the breaker of rocks for foundations, the ashlar squarer, the sculptor (who worked above all on white marble: the *marmararo*), the polisher.

Both stone, especially marble, and wood, in the form of planks and beams, required reduction operations by means of sawing on the site; these were performed by *segatori* or sawyers, sometimes on new materials and sometimes to adapt recovered materials for a new use.

The *fabbro* (smith) would appear to be a secondary figure if we think of the use, fundamental but limited, of iron in masonry structures. Instead, his role was constant and basic for the site: he produced every tool with metal components and kept it efficient.[32] He acted on them with steeling (hardening of the surface through the addition of carbon), sharpening and remaking of blades or tips, welding of broken pieces.

The smith also prepared a whole series of special pieces, required at different times, for attaching fittings to walls (*trumeau,* mirrors, etc.), for supporting stucco decorations or for kitchen needs: parts of hearths, and gratings to put at the windows or elsewhere. A noteworthy part of his work was producing nails: up to 35 different types have been found.

For placing *crowns* —the iron tension rods that oppose the horizontal thrusts of arches and vaults or tie masonries together— the smith had to prepare the pieces starting from semi-finished parts[33] that were supplied to him, assemble and connect the various elements and lastly tauten the chains; each f these operations required fire.

Some specialisations in metalworking are that of the *ciavonero*, who dealt with keys, locks and hinges, and that of the *latunaro,* who prepared lead pipes and brass taps.

The *bancalaro* or carpenter played different roles going from the realisation of substantial parts of the construction like floors and roof structures, to finishes like doors, windows and shutters and fittings. The latter work, especially in the case of buildings over which particular care was taken, was part of the site in timing, and was already being done when the edifice was being finished, also because of the relation between architecture and interior design.

If this was quite evident in the eighteenth century, a period in which there was continuity in the decoration of walls and furniture, in previous centuries too a contribution which was not negligible was made by the various built-in wardrobes, some of them hiding toilets, closed off by panels of wood, often decorated or carved, and the participation in the architecture of a whole hierarchy in the doors, from the most decorated ones leading into reception rooms to the simplest ones leading into service rooms, some of them carefully chosen, in the materials and finishes, so as to withstand thieves and damp better.

The carpenter, like the smith, was constantly engaged in the construction and maintenance of tools and other things necessary for work, like for example the compasses used by other artisans or the handles of various instruments.

In some operations, like attaching door hinges or systems for supporting various construction elements, the stonemason and the smith worked together: the stonemason prepared in the stone the lodging in which the smith positioned the object and embedded it by pouring in lead.

Lastly, there were craftsmen not strictly belonging to the building world who nonetheless played a role on the site too, for example the *bottaio* (cooper) and the *vetraio* (glazier). The cooper provided the buckets, pails and vats used for the slaked lime. The glazier prepared the window frames, both with lead and, starting from the 18th century, with wood and putty, as well as selling *conche*, *corbei, giare*, *bottiglie*, *trombette* . . . i.e. the various glass recipients and glazed earthenware objects, to be used above all by *stuccatori* (stuccoers) and *pittori* (painters). *Calderari* too (i.e. boilermakers), normally busy with saucepans and boilers, were involved for some jobs with copper and brass.

Processing on the site

Almost all building materials require processing on the site before or after being put in place, either because of the nature of the material itself (as in the case of lime, which has to be slaked) or because transportation might damage pieces (as in the case of marble or bricks), or because of adaptation to measurements and sections (typical of wood and iron), and of course because many surface finishes can only be done as the last operation (polishing, colouring . . .). In light of the widespread practice of reemployment of materials, we also have to consider all the adaptations of the recycled material.

Starting from the 16th century, the stones used in masonry structures were supplied, usually on a daily basis, in various sizes, but were only put in place after selection. At most the mason doing the wall intervened for small adaptations, but his ability consisted precisely in the capacity to assemble an apparently very regular device that yet proved to be very solid (Mannoni 1997). A different case was that of masonries with squared-off ashlars, typical of the previous centuries, which required the patient work of the stonemason to realise the six faces of the parallelepiped which was afterwards to be put in place without particular expedients, unless there was smoothing of the outside face, to be done after assembly.[34]

Stones for finishes or for the decorative parts of structures, like stairs, colonnades and balustrades, balconies, floors, covering surfaces, cornices of apertures and plant were instead supplied in pieces that were more or less semi-finished parts.

In the case of columns, monumental portals or elements like chimneys, whether in Carrara marble (extremely widespread in Genoa architecture) or Lavagna stone, a local slate, each piece was ordered to size; sometimes the system worked even for the exportation of elements abroad.

By contrast, little balusters (in marble), door and window cornices and stone flooring elements (in marble or slate) were produced in a more standardised way, making them to some extent datable on the base of dimensional ratios. The supply of pieces of standard length (Decri 1991), already shaped (*alla romana*, *piani*, *lavorati in faccia*, *refilati*) made adaptation necessary before they could be put in place.

Mention was made above of the work of sawyers on stone and wood materials. It can also be added that in cutting marble, usually a smooth-bladed saw was used with sand acting as an abrasive, and that this instrument needed continual maintenance because it wore out.[35]

Floor bricks needed to be made on the site for perfect fitting, with very thin joints, which could not be done before because of the risk of ruining the edges in transportation. We are referring to *squadratura e fregatura di quadretti* (squaring and cutting of little squares). It is shocking to consider the number of such precision jobs: tens of thousands on a single site.[36] Probably after they were put in place the last operation on them was polishing, an operation which could be performed with rasps and then *arenino*, used as a fine abrasive.

NOTES

The divisions «Workers» and «Processing on the site» are written by Anna Decri, the others by Anna Boato (Dipartimento di Scienze per l'Architettura, Università degli Studi di Genova).

1. Archive documents here used come from a database made by the authors during the following researches: *Fonti scritte e fonti materiali per l'edilizia dell'età moderna, Storia dell'uso dei materiali edili a Genova, Tecniche costruttive, manutenzione, materiali, restauri: il caso ligure,* 1988 - 1996 Facoltà di Architettura di Genova (Genoa operative unit coordinated by prof. L. Grossi Bianchi). Researches on the argument of this paper was made in: A.Boato, *Costruire a Genova tra medioevo ed età moderna*, tesi di dottorato in Conservazione dei beni architettonici, VI ciclo, 1995; A. Decri, *Conoscere l'architettura, manufatti nel Settecento genovese*, tesi di dottorato in Conservazione dei beni architettonici, XII ciclo, 2002.
2. Till now data have been processed relating to two hundred 15th-century and 16th-century contracts.
3. Archivio di Stato di Genova (A.S.G.), Notai Antichi (N.A.), 1032, 1–6–1491.
4. A.S.G., N.A., 1292, 1–6–1489.
5. A.S.G., N.A., 1305, 9–7–1499.
6. A.S.G., N.A., 5829, 7–5–1623.
7. «Intendendo che tutti li zetti ch'avanzerano de rempire le volte della detta fabrica si debano distrebuire nella detta villa» (A.S.G., N.A., 2548, 7–11–1562); «fare portare via parte de quello zeto l'altra parte lasarlo soto l'astrego de deta ciostra» (A.S.G., N.A., 2552, 1–3–1566); «Le terre che si caveranno per questo novo apartamento han da servire per apianare il giardino che si è fatto in pian de saloti di mezo giorno e doppo definito questo. Il resto che avanzerà farà portare verso la villa alli lochi dove parrà più bisogno e a proposito» (A.S.G., N.A., 4533, 7–4–1614).
8. «Le terre bone (cavate dai fondamenti) servirano per fare matoni. Li gietti e terre cative si giteranno in mare» (A.S.G., N.A., 5987, 14–12–1629).
9. «Dictosque lapides dietim secundum fabrice indigentium consignare in locis propinquioribus dictæ fabricæ et illos a canella suis expensis congerere et, ut aiunt, acanellare facere. Prætio et mercede [scilicet] illorum a canella librarum sex et [solidorum] 12 singula canella congesta ut supra» (A.S.G., N.A., 5963, 17–9–1619).

10. «Portare le pietre da canella sopra li ponti alli piedi delli maestri in quelle parti dove travaglieranno et le pietre piccate condurle alli piedi delli pilastri dove sarà ordinato e poi a tirarle sopra li pilastri dove li maestri muratori haveranno fatto le cravie per tal'effetto» (A.S.G., N.A., not. Bartolomeo Castiglione, 14–12–1660).
11. In the accounts of Palazzo Gerolamo Pallavicini mention is made of «Un rastello di ferro per uso della fabbrica per la calcina» (Archivio Pallavicini II, 43, 366, January-December 1720). The term *rastello* means «gate». So the *rastello* in question is probably the gate or grid that is placed between the slaking tub and the seasoning pit, for sieving the *crudi*.
12. «Che detta calsina sia fresca, bona e mercantile . . . e che il peso sia sempre fatto nella fabrica dal soprastante nè altrove nè da altri, il quale, giudicando detta calsina bona e della qualità sud.a conforme il peso da esso fatto si debba pagare a ragione di lire 12 il moggio, detraendo però il crudo, con dichiaratione che quando che non vi ne sia più de rubi 4 per moggio non si debba detrahere nel pagamento il quale crudo di più che vi sarà non si debba pagare e doverà esser conosciuto dal soprastante» (A.S.G., N.A., 5639, 8–9–1627).
13. «Stagno consumato in aggiongere li canali nel vicolo delle Merini, ad effetto di fare venire l'acqua per stemperare la calcina nella fossa» (Archivi Pallavicini II, 43, 471, 25–4–1722); «aqua segie n° 45» (A.S.G., N.A., 5658, 6–3–1638). «se al posso vicino a d.a fabrica non vi fusse aqua sarà detto maestro Michele obligato fargliela portare al piè della fabrica» (A.S.G., N.A., 6023, 26–4–1643); Bruzzo 1935, 24–25.
14. «4 donne che hanno bagnato la calcina —datto a uno chi ha sciorato la calcina» (A.S.G., N.A., 5644, 14–6–1630).
15. «Far portar via la terra che sopravvanza al suolo dove adesso è la fossa della calcina a segno che sii a piano dove è detta fossa» (A.S.G., N.A., not. Gio Lorenzo Assereto, 1688).
16. «Fare la baraca per metere la calcina et per impastarla» (Ghia 1999, 304). On the same site there was also a shack for the stonemasons (Ibidem).
17. «Il Magistrato darà comodità di stanze o baracche per li operai e per le biscazze» (Bruzzo 1935, 48)
18. The regulation is inserted in the «Ordini della Camera dei Prestantissimi Signori Padri del Commune» of 15 July 1594 with the title «Non si faccino ponti in strade publiche senza licenza. Con licenza siano ben sicuri» in Archivio Storico del Comune di Genova (A.S.C.G.), Atti dei Padri del Comune, 58.
19. An abundant series of decrees regulated the cleaning of streets, the removal of rubble, the emptying of *gabbioli* (deposit pits) existing at the estuaries of streams and of sewers, and works for containing cultivated lands present in the whole hydrographic basin (Desimoni 1885).
20. Rich pictorial documentation can be found for example in Du Colombier 1973; Goldthwaite 1984; Caniato and Dal Borgo 1990. A work wholly devoted to scaffolding is AA. VV. 1996.
21. The palace, in Via Garibaldi, was built between 1571 and 1578. We can mention also the numerous scenes on building subjects depicted in Villa Musso Piantelli at Marassi (beginning of the seventeenth century).
22. «Ha accomodato 17 canteri da 100, 37 tavole da ponte e alquante fascine e una coffa con il cormo alto piena di corde che doverà restituire infine» (A.S.G., N.A., 5570, 19–7–1645).
23. We find mentions of supporting structures either in contractual clauses, «se imtende che il detto m. Domenico debia provedere per dicta opera de tutti li ferramenti laborat [. . .] coffe legnami ceitri tavole di armari et altre cose che si bizognano per dicta opera», (A.S.G., N.A., 1905, 23–10–1545) or in the lists of expenses: «tabule per fare li cetri dopie e sempie [. . .] aguti per li ceitri grandi e mezani» (A.S.G., N.A., 1905, 5–6– 1545).
24. Varni 1877, 14–16 and table 1, also reproduced in Ghia 1999. The memorial is dated 30 November 1560.
25. «Tagias arganos et cavos sive agumenes pro tirando bordonaria et lignamina» (A.S.G., N.A., 832, 23–10–1454).
26. «Le pietre piccate . . . tirarle sopra li pilastri dove li maestri muratori haveranno fatto le cravie per tal'effetto . . . Similm.te s'obbliga di farle provedere di legnami per far li calandroni e cravie per tirar le pietre piccate e similmente taglie e cavi e argani necessarij per detto lavoro e non differentemente e similmente provederle di corde per fare li calandroni e chiodi se ne farà bisogno a giudicio del capo d'opera della fabrica». (A.S.G., N.A., not. Bartolomeo Castiglione, 14–12–1660).
27. A.S.C.G., Cartulari dei Padri del Comune, 9, 20/12/1471: registration of a payment «pro expensis factis in legias pontes lignamina et alia opificia excogitata ac designata a dicto magro Anastasio». The same source also mentiones various tools for lifting («pro tagijs, pulegijs, pastechis»). The term *taglia* is still used to indicate a lifting machine made up of fixed and mobile pulleys; the *pasteca* is a sort of *taglia* used above all on ships (Casaccia 1851).
28. «Pro circulis 4 magnis ad faciendum rotam in albario; pro caveto uno cantariorum trium et rotulorum 25 pro rota ponthoni» (A.S.C.G., Cartulari dei Padri del Comune, 9).
29. For example in the 17^{th} century there were 64 guilds, some of them (*bancalari*, *ferrari*, *muratori*, *serratori di tavole*) peculiar to the building world, but various workers came under the same guild. Cf. A.S.C.G., Magistratura dei Padri del Comune, Arti.

30. «Per fare detto lavoro Bartolomeo vi metta buoni maestri senza garzoni e che al più vi ne sij uno del sotto capo de opera». (A.S.G., N.A., 5137, 8–8–1629).
31. We get an idea of the organisation required by a big site like the Carignano basilica one from reading «Regule e ordini sopra la fabrica»: this document precisely establishes the organisation of the workers on the site and the tasks assigned to each of the people responsible for the financial management and the supervision of the works: the Magnificent Gentlemen Executing the Fabric, the two Deputies of the Executors, the Architect, the Clerk, the Cashier, the Overseer (Ghia 1999, 295–300).
32. The quantity, shape and use of the tools employed by all the craftsmen present on the site still remain to be investigated in depth, one reason being that not all the construction techniques are known. This is a first list derived from the documents examined: *agoglie, punte, picche, picchette, marapiche, picconi, picozzi, martelli, frapine, badili, zappe, cunei, frappi, martelli, mazzuoli, scalpelli, trapani, tenaglie.*
33. «Fare e consegnare chiave greze a soldi 24 il cantaro di manifatura, chiave di ferro polito a soldi 32 il cantaro di manifatura, chiave vechie rifare accomodare et agiongere a soldi 32 il cantaro, vele di ferro per il tetto di manifatura a soldi 50 il cantaro, e più ferrate grande e piccole di manifatura a soldi 50 per cantaro, connij di ferro da requadrare pilastri et altro di manifatura denari 8 la libra. Connij di ferro da requadrare pilastri et altro; ferro e manifatura in tutto soldi 2 e denari 8 la libra con patti che d.i lavori siano ben fatti politi et uniti insieme senza crepature e forti di quello istesso ferro che per questo effetto li sarà dato e caso che si rompesseron così prima come dopo d'esser messi in opera per colpa delle manifature perché fusseron in qualche parte mal fatti debba rifarli, et il peso sia riconosciuto dal soprastante». (A.S.G., N.A., 5639, 20–9–1627).
34. «Lapides esse debeant undique quadrati et plani, et cum pacto quod si postquam essent positi in labore protenderent et excederent rectum muri teneatur d.s m.r Nicolosius et in labore aptari et asplanare facere ac reducere ad rectum muri». (A.S.G., N.A., 5965, 22–8–1620).
35. «Comodato una sera per il seratore de marmari adrizzata a caldo per essere frusta al mezo e tagliato al mezzo altra e di nuovo gionta. Frazattura di sega et arena in più volte: palmi 142». (Archivio Durazzo, 478, 1755–1757). *Frazzà* means decrease, disperse, drop, diminish, be partly missing, consume (Casaccia 1851).
36. «Per scuadrare n° 9600 quadretti solo di quadratura per essere statti fregati in gta a conto de suddetto ill.mo signore a ragione di soldi 30 il centinaio £ 144 per n° 27300 sudetti quadretti scuadrati e fregati a scarso a soldi 52 il centinaio £ 709:16 per scuadrare n° 900 quadretti vechij a soldi 16 il centinaio, £ 7:4». (Archivio Durazzo, 477, 8–2–1755).

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Masonry vaults in Genoa: From historical and archaeological analyses to scientific interpretation of the rules for their construction

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Sergio Lagomarsino
Daniela Pittaluga

In the course of successive investigations conducted by the authors,[1] much data relative to the construction of masonry vaults was collected, particularly with regard to the city of Genoa in the modern era (sixteenth through eighteenth centuries). The sources of information utilised were two: the so-called «material source», comprising the structures that are preserved and can be inspected in one way or another; the «written source», comprising the documents in archives. In the course of this and other research, it was possible to observe how a systematic and careful comparison of the data gathered from two sources permitted us to better understand and interpret the data itself.

With regard to the material source, 120 vaulted structures were analysed. These analyses, which we define as archaeology because they were conducted according to the principles and methods of building archaeology, took into account the following aspects: general characteristics of the structure and its dating (morphological and typological analysis, stratigraphic analysis, chronotypological analysis of individual architectonic elements); materials utilised and their dating where possible (archaeometry of the mortar, mensiochronology of the bricks); the construction techniques adopted (general technological analysis, analysis of the masonry technique).

With regards to the written source, we availed ourselves of a digital database begun at the end of the 1980s that gathers more than 700 notary contracts concerning the building trade. For the purposes of research of vaulted structures, of particular interest were the documents of construction describing verbally, sometimes in more and sometimes in less detail, the work to be executed, whether in the case of new construction, restoration or maintenance of existing structure.

Given the differences in nature between the two sources utilised, the information that they provided was in part similar and directly comparable. Many times they completed and added substance to each other reciprocally. Sometimes they indicated the necessity for new research to fill gaps in their respective areas of applicability.

Genoese vaults of the modern epoch

Geometric form

The most frequent typologies turned out to be those of barrel vaults, cross vaults, pavilion vaults and barrel vaults with pavilion vaults at their ends (which frequently had lunette vaults all around the perimeter).

Material form

The most frequently used material (in about 90% of the cases) is brick. Comparing vaulted structures to

other, coeval, masonry structures, several significant differences were noted: 1) in vaults, within the same structure, there were very frequently two kinds of bricks used, wall bricks and brick pavers, while in walls this combination was much less frequent;[2] 2) the bricks used in vaults were characterized by a much greater[3] dimensional homogeneity than that found in walls. This fact indicates the accurate selection and verification of the quality of the product undertaken by the builders at the time the material was acquired; 3) in vaults, contrary to what happens in walls, there is a sporadic use of recycled brick; 4) with respect to walls, it is possible to note a more frequent use of bricks that are very well fired (the so-called *ferrioli)*, and vice versa, the scant presence of bricks that are not very well fired (these, when present, are mostly concentrated in the area at the centre of the vault).

Thickness

The thickness of masonry vaults in residential buildings, extracted from documentary information and verified by data gathered in the field, is generally a half-palm (approximately 12.5 cm),[4] a dimension corresponding to the width of the brick. In the zone of the imposts, however, the thickness can double. Further, continuous or interrupted ribbing at the extrados is often provided (formerly called *ghiane* in Genoa), in correspondence to which the resisting structure doubles its thickness.

Buttressing and infill

Both the documents and the analyses of existing structures testify to the use of brick or stone buttressing set with mortar (the so-called *masicci*) to stabilise the vault. An analogous role was played by the infill with rubble. In the case of small- and medium-sized vaults, rubble completely filled the spaces above the vault and, appropriately levelled, provided the setting bed for the pavement above. In the case of larger vaults (in churches and in the great halls of palaces and villas), the excessive load of a complete infill was avoided by means of structural solutions (small walls or vaults) or, where possible, allowing the top of the vault to emerge into the space above (usually attics or the spaces under the eaves).

Chains

The archaeological analysis of existing structures has verified the use of metal chains that are visible in the intrados of only 19% of the cases examined. Further, the presence of chains has been verified in the extrados of another 15% of structures. It is probable that, if not in all then in a great part of the remaining vaults, there is some form of hidden concatenation (within the precinct walls or in the extrados), in accordance with what is found in the documents. Even though the use of chains was not considered indispensable by builders in the past,[5] in the contracts the placement of some kind of chain is often explicitly provided for, whether simple or furnished with *bracci* (literally, «arms» or oblique branches), sometimes arranged according to a kind of frame.[6]

A CASE STUDY: THE VAULT OF SANTA MARIA DELLE GRAZIE

Among all the vaults studied, of extreme interest was the barrel vault with lunettes and pavilion vaults at its ends, Figure 1, that is found in the former convent of Santa Maria delle Grazie la Nuova.[7] The convent was built starting in the fifteenth century on a site on which there had already been intense building in the Middle Ages. Between 1385 and 1460 the nuns proceeded to acquire various existing houses, with the aim of building «a church and convent with a cemetery and bell» dedicated to the Madonna delle Grazie (Boato 1997). A substantial part of the medieval walls was embedded in this new construction. They are still visible in part on the south side of the church. In the almost four centuries that passed between the first settlement of the nuns and the closing of the convent following the Napoleonic disposition in 1810, the complex was the object of continual construction campaigns, sometimes quite extensive.

In particular we know of a significant restructuring begun in 1623 (Costa 1934 gennaio: 3–5). In that year the nuns presented their request to Pope Gregory XV to be granted the funds required to enlarge the complex to make it adequate for the use of a religious

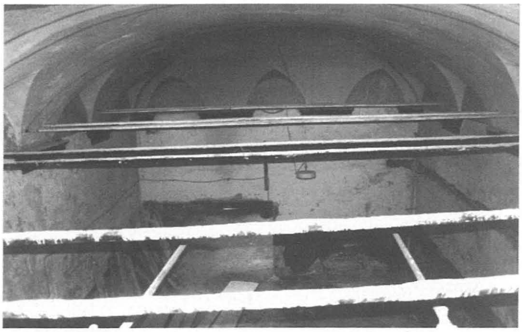

Figure 1
Intrados of the vault. The couples of intradossal chains are shown

community that had by then grown to more than a hundred people. In the memos they set forth the necessity of amplifying the refectory and the work places, to increase the number of living cells and to enlarge the small *ecclesia interior*, then no larger than forty palms and not wider than fifteen palms (9.19 meters by 3.72 meters), to the more decorous fifty-four palms in length by thirty palms in width (13.38 meters by 7.43 meters).

Almost certainly, the result of this campaign is recognisable in the ample space to the north of the church, the end wall of which is richly decorated with stuccoes and which is covered by the vault that is the object of our analysis. The dimensions of this space (about 15.0 meters by 7.10 meters) are not in fact very different from those envisioned at the time, thus a space of dimensions that correspond to those of the previous «interior» church can be obtained by projecting onto the ground plane the structure of the walls above, which on the upper floors delimit the spaces that have characteristics that are more archaic than those of the lower hall.[8]

It is in fact the presence of walls of notable thickness and height bearing on the vault both longitudinally and transversally, together with the hypothesis that the realisation of the vault took place during a daring operation of centering, demolition and underpinning, that suggested the idea of a study of this structure. It symbolises all that the mastery of Genoa at that time could realise, thanks solely to the knowledgeable application of the construction methods conventionally used.

Even at first glance one notes an exceptional abundance of chains that tie in the vault transversally slightly above the level of the impost: there are in fact ten, placed in pairs in correspondence to every corbel. In addition this already remarkable system of concatenation, thanks to the degradation of the plaster, it is possible to see the presence of other,

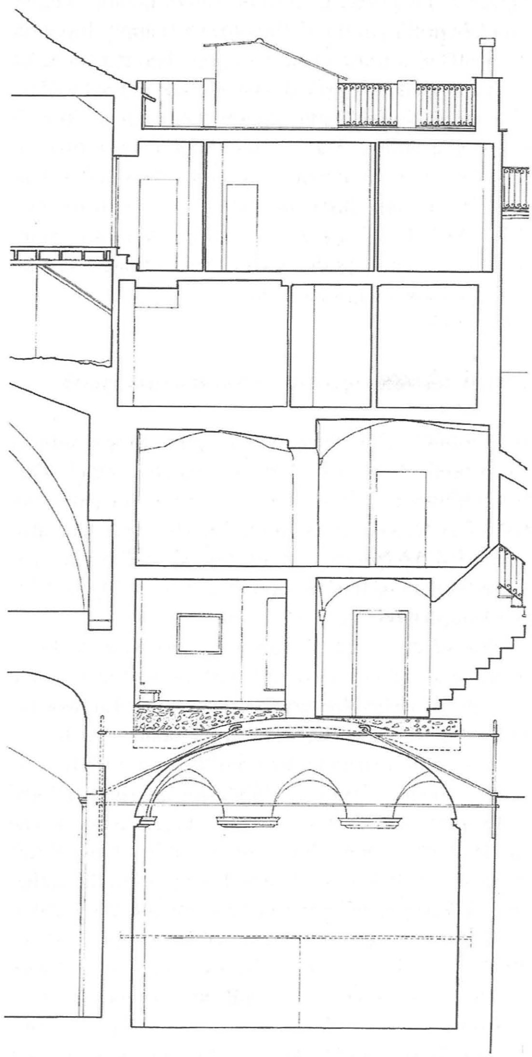

Figure 2
Transverse section that shows the extrados chain with its oblique branches and the intrados chain

hidden, chains that intersect the vault at the intrados, in correspondence with the apex and on its sides. This is a «slinging chain» system, usually employed as an alternate rather than in union with chains at the intrados. The anomalous adoption of a double system of concatenation is undoubtedly motivated by the exceptional nature of the context. As already noted, a wall some 75 cm thick and originally at least 6.5 meters high rests right along the apex of the vault, Figure 2. Two other lengths of wall of notable weight (one about 85 cm thick, the other 50) apply force on the vault in a transversal direction. The existence of such a load must weighed on the minds of the builders as well as on the vault, urging them not to stint in taking precaution. There is no doubt, in fact, that the whole system of chains was conceived and set in place at the same time that the vault was constructed. The proof of this lies in the fact that the upper chain, the lower chain and the diagonal branch are fixed by a single long anchor element.[9]

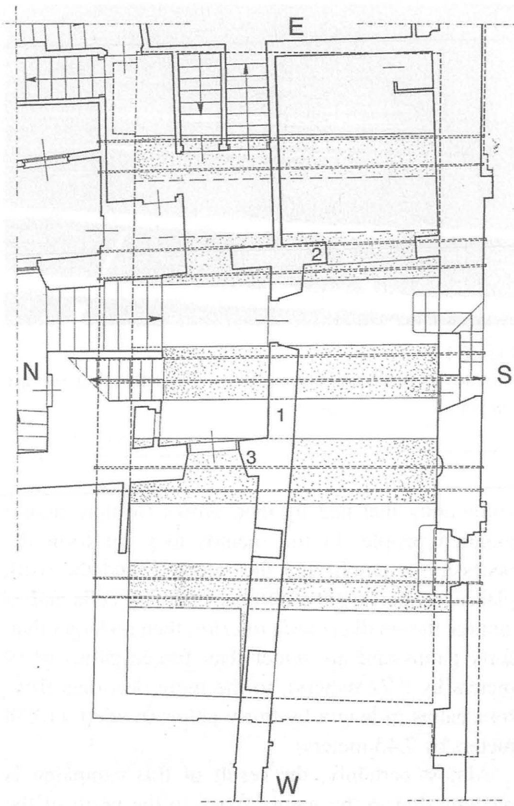

Figure 3
Plan of vault extrados. This picture shows the five transverse arches and the five couples of extrados chains. The line of the walls at ground level is outlined too

Investigation into the construction phase

Beginning in 1994 the edifice in question was subject to a careful restoration, still in progress today. This has permitted the inspection and survey of parts that would otherwise have been hidden, and has also permitted the carrying out of several studies aimed at comprehending the construction system adopted for the vault.

The extrados of the vault is covered with rubble.[10] Some excavations have allowed us to see that the extrados is ribbed by means of a system of transverse arches of widths that vary from about 1.0 to 1.5 metres, with a spacing between them of 1.05 to 1.45 metres. Such variations in dimension are surely related to the presence of the walls above, Figure 3 and 4. The arches have a thickness at their centre of two brick headers, equal to about 25 cm. The bricks of the arches do not appear to have ever been keyed into those of the vault below, the thickness of which is equal, at the vault's centre, to 27.5 cm. However, one link between the two structures is shown by the presence of numerous large slabs of slate, which, in all probability, regarded the entire thickness of the vaulted structure.

In correspondence with the longitudinal wall, the head of the pavilion placed at the western end of the vault is likewise ribbed by a climbing arch on the extrados, which on one side has its impost in the perimeter wall and on the other is keyed into the side of the transversal arch. The thickness at the centre of the climbing arch is also equal to two brick headers, while it increases as it nears the impost.

Between rib and rib, in correspondence with the longitudinal wall, have been built very shallow arches, two brick headers thick, which have the function of counter-bracing the ribs and the transfer of the loads of the part of the wall above, Figure 5.

The transversal arches are fitted with siding of brick and stone with lime mortar, the summit of which is at the same height as the chains of the extrados, Figure 6. Thus the part of the chain that

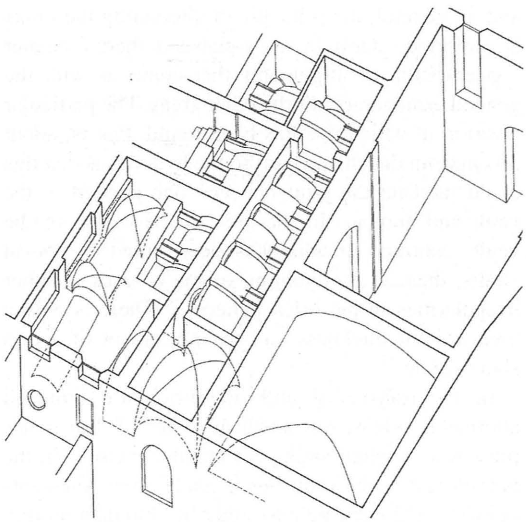

Figure 4
Axonometric reconstruction of the vault structure

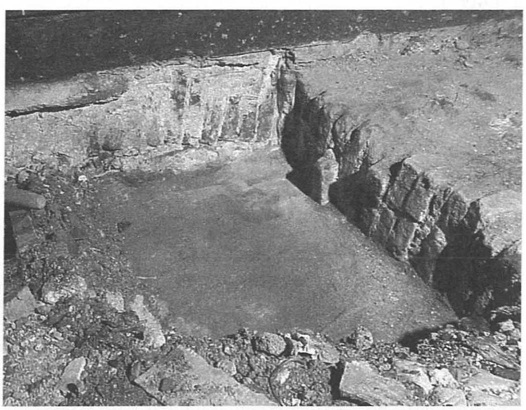

Figure 5
Extrados vault. Detail of the binding between a transversal arch and a little longitudinal arch that binds together the transverse arches

touches the extrados remains firmly fixed for its entire length and is more protected than it could have been by a siding of loose rubble. However, it isn't clear whether this construction system is adopted throughout or not, because it has been only been possible to affirm this is one of the two studies that it has been possible to carry out. In the other study only a simple accumulation of rubble was found. The vault is fitted with mortar siding as well.

Figure 6
Extrados of the vault. The following elements may be recognized: the transversal arch, the extrados chains, the climbing arch at the end of the vault

Comparing the position of the transversal arches, reconstructed point by point thanks to the investigations performed on the extrados, with that of the chains that are visible on the intrados, it has been possible to verify, as was expected, a good correspondence between the dimensions: the five pairs of upper and lower chains therefore contain the thrust of the vault precisely in correspondence with the arches, which constitute in all senses therefore the principle load-bearing construction system.

Characteristics, preparation and setting of the materials and the elements of construction

Observing this structure raises the question of whether the same investigation that informed the

whole process of design and construction of the project finds a correspondence in the choice of materials and the setting in place of the elements. The answer is affirmative: in S. Maria delle Grazie the careful selection of the materials is associated with their accurate use. Slight anomalies appear to have been dictated more by the necessity of adaptation to pre-existing conditions than to negligence on the part of the builders. An analysis of the materials employed gives rise to the idea that there was a careful selection process.[11] The homogeneity of the bricks, for instance, is superior to what can be observed in other similar structures of the same period; the elements in stone that are used appear to be all of optimum quality. The mortar causes some perplexity: it contains some lime gravel[12] and doesn't have any particular qualities of cohesion or adhesion.[13] The system of underpinning and its relative construction in parts requires an accurate operation that must be executed by able master craftsmen: the analysis of this structure has revealed an elevated degree of competence on the part of those who executed the work.[14]

The bricks

In terms of dimension, colour, and the characteristic of their mixture, the bricks used in the vault, the transversal arches and the connecting arches are very homogenous.[15] These could be those which in the documents are called *ferrioli*. Very rarely, and only in the supports, are there bricks that are recycled.[16] Relatively scarce, with respect to the construction norms adopted, is the use of brick pavers.

In the vaults the bricks are arranged prevalently head to head, and rarely side by side.[17] Of particular interest is the information regarding the frequency and the variation with which these cases are present: in the centre of the area comprised between the two chains are 29% of the bricks arranged side by side; this figure drops to 17% in the remaining areas. This last figure, however, is characteristic of the overall brick pattern, while the greater resorting to laying the bricks side by side in the area between the chains appears to be explained by the greater ease of laying them this way and by the necessity of covering the chains of the extrados that intersect the intrados at the centre of the vault. The brick pattern is very regular and, in general, the principle of alternating the joints is respected.[18] Only in some places is there a greater concentration of alignments that contrasts with the general arrangement of the other areas. The particular position in which this has been found, that is, about 30 cm from the chain, leads to the hypothesis that this could indicate the joint between one portion of the vault and another. In the area of the centre of the vault, contrary to what has been noted in coeval vaults, there is no doubling of the courses or other irregularities in the brick patterns.[19] There is only a greater joint thickness and the placement of a thin slab of slate.[20]

In the transversal arch the brick are arranged alternately side by side and head to head.[21] The arches present a coherent siding for about a metre:[22] in the part closest to the vault this is made up of fragments of bricks and stone held together by abundant mortar. In the upper part the bricks are prevalently arranged in an orderly fashion: »di piatto» (horizontal alignment) in the parts near the perimeter wall; «di coltello» (vertical alignment) in the remaining part, Figure 7.

The shallow longitudinal arches are entirely of brick, with no insertion of stone, an expedient that is usually necessary in case of filling and adherent construction. There are some slight variations in the joint thicknesses. This too could be an indication of the care with which this structure was built.

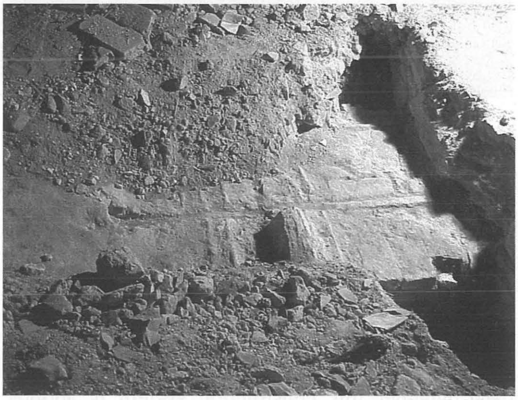

Figure 7
Support of the transverse arch

The stone

There are few stone elements inserted into the vault. They are essentially constituted of slabs of slate of the best quality, with a thickness of between 5 and 12 centimetres, and of coarse workmanship about their edges.[23] Probably only partly dressed, they were used here in the same form in which they arrived on the construction site. On the other hand, their characteristics turn out to be optimum for creating a valid key («tie») between the vault and the large arches above.[24] In the case of the smaller slabs, rather than having a single element in the arch and in the vault, there are two elements one above the other, one in the arch and an analogous one in the vault. These ties are rather frequent: on the average every 25 brick courses. These elements are placed in any case at the points of the greatest force (for example, in the centre in correspondence with the wall, between the first transversal arch and the reinforcing arch, etc.). The large slabs are always, in the sample area, arranged obliquely.

A RECONSTRUCTION OF THE SYSTEM OF UNDERPINNINGS USED

An archaeological analysis of the elevation was fundamental to understanding the sequence with which the complex operation of underpinnings was undertaken in order to allow the realisation of the vault beneath a pre-existing wall. In particular, the stratigraphic analysis based on the observation of the interface between the various elements (vault, transversal arch, longitudinal arches), of the mortar joints, of the presence of chips or scales permitted a reliable determination of the successive operations undertaken. For example, in order to determine the sequence between the transversal arch and the vault an examination of the mortar turned out to be most useful. The mortar bed of the vault, spread on its extrados, contains the imprint of the bricks of the arch above. On the other hand, the mortar of the arch presents a clean continuity with respect to that of the vault on which it rests. The arch is therefore stratigraphically posterior.[25]

In the longitudinal wall and transversal wall 2 can still be seen some elements that, in all probability, must have been employed in the underpinning operation. Wall 2, precisely in relation to that operation, was emptied with a large opening in the form of an arch. In correspondence with the left side of the opening, at approximately 1.2 meters from the apex of the arch of the extrados, there are still visible some wooden wedges and a piece of iron. The wedges have a reduced thickness (a maximum of 2 cm), and are approximately 10 cm wide and 20 cm long. They are arranged in three series: two next to each other, with five elements one on top of the other; one, with only two elements, that abuts the opening, Figure 8. The iron bar, inserted in this last portion of the wall as well, is some 2 cm thick and 7 cm wide. At the moment its length cannot be defined, but it is surely of notable dimension. The iron bar and the series of two wooden wedges are placed between two blocks of limestone;[26] in correspondence with the two series of five wooden wedges, one above and one below, there is instead a layer of mortar about 2 cm in thickness. Between the two series of five wedges and the ribbing of the vault is placed a wooden mounting with a circular section of about 20 cm in diameter, inserted within the masonry.[27] The mounting and the wedges are of different kinds of wood.

Wooden elements (wedges?) and some small iron bars[28] are also inserted perpendicularly in the longitudinal wall. They are placed at different heights with respect to each other, varying from 70 cm to 123 cm with respect to the extrados of the transversal arches. The different levels of these

Figure 8
Transversal wall 2. The wedges used for the underpinning are shown

elements should perhaps be seen in relation to the portions of the wall to be demolished, which turn out to be different depending on whether there is a correspondence to the area in which the arch that was above the vault was to have been constructed, or of the area in which only the vault was to have constructed. It is possible that an accurate study of this could furnish interesting information regarding the dynamics of the underpinning operation.

Currently being studied is the role played by the small wooden beams, the heads of which can be seen projecting along the wall at a level of approximately 50 cm from the extrados of the arches, Figure 9. These also seem to be connected to the underpinning operation.[29]

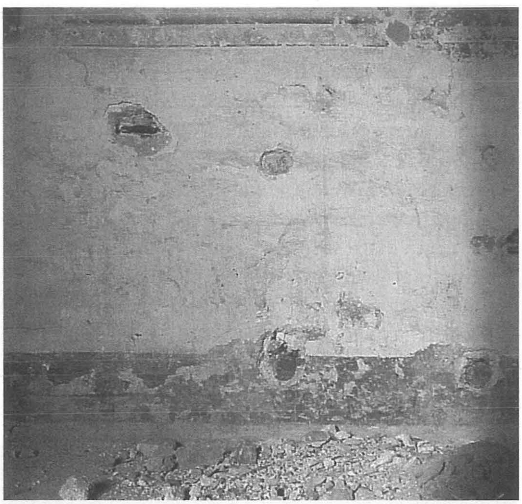

Figure 9
Longitudinal wall built on top of the keystone of the vault. It is possible to recognize the heads of two wooden beams and two iron bars

The underpinning operation was then undertaken thus:

1. the partial demolition of the wall (of a dimension necessary to permit the insertion of a single arch of 1.0 to 1.5 m);[30]
2. the realisation of a portion of the vault (leaving the ties as required);
3. the realisation of the first transversal arch;
4. the resting of the wall on the first transversal arch by means of the filling of the residual opening;
5. the partial demolition of another part of the wall;
6. the realisation of a second portion of the vault (with the necessary ties);
7. the realisation of the second transversal arch;
8. the resting of the wall on the second transversal arch;
9. . . . a repetition of this same sequence for each of the transversal arches;
10. the demolition of the portions of wall between arch and arch;
11. the realisation of the shallow arches that connect the various transversal arches;
12. the resting of the wall on the shallow arches.

The most delicate aspect of this type of operation is due to the settling that can eventually occur at the end of the operation. At this time, given the results of the archaeological analysis, there does not appear to have been any particular problems at the time of the construction campaign.

Delimiting the construction campaign

The operation at the beginning of the seventeenth century described above is in all probability tied to the construction, or reconstruction, of the adjacent chapel, where, in 1631 the ashes of the Venerable Battista Vernazza, abbess of the convent who died in 1587, were transferred.[31] The longitudinal wall weighs on this space (measuring 5.25 by 5.75 metres) as well. The archaeological survey of the façade, executed before the actual plaster was set, indicates a series of small wooden keys that correspond to the height at which the longitudinal wall intersects, on the interior, the vault below. On the façade the keys turn out to be positioned at the lower limit of a tract of wall comprising square stone ashlar, a masonry device that is typical of the medieval. In the portion of the wall below, in any case, is found a mixed kind of modern masonry, that is, of a later epoch. The mensiochronological and mortar analyses conducted on several points on the interior confirm the hypothesis, placing the construction campaign

securely within a period[32] contemporaneous with that of the vault that was the subject of the present study.[33]

The fact that the two campaigns, the one regarding the space covered by the pavilion with lunettes and the other relative to the chapel, were contemporaneous partly explains the double concatenation of the wall on which the vaults of both weight. This wall contains a relieving arch that is analogous to the transversal arches of the vault, which can also be connected to the method of underpinning. A first pair of chains is situated at the impost of the arch,[34] its anchor element is visible. A second pair of chains, buried in the thickness of the wall at approximately 2.0 metres above the ground level, cross the entire length of the wall and insure the integrity of the longitudinal walls. These are similar in their fabrication and dimensions to those characterizing the whole of the campaign.[35] This concatenation of the surrounding walls can be found only on this side and does not appear to be present on the opposite wall.[36] It is possible that this difference is not coincidental but is rather tied to (empiric) considerations made at the time on the different structural roles played by the two walls. Even if the insertion of the chains in the surrounding walls of the vaulted spaces is provided for in the documents, in reality it turns out to be of scant practicality.[37] Its use in this construction campaign could therefore be consistent with the desire for security that is evident in the project as a whole.

THE STRUCTURAL ANALYSIS OF THE INTERVENTION

The construction of the vault, built under the pre-existing medieval wall, is an example of great interest both from the historical and technological point of view as well as from the structural one. This architectonical element allows us to understand the true capacities of the arch structure. It is obvious, in fact, that the rule-of-thumb methods and the criteria of geometrical proportion could not be directly applied to such a particular problem, without a knowledge of the structural behaviour.

The builder's worries for such a delicate and unusual intervention led him to adopt a hybrid structural solution for the chain system. Besides the traditional «slinging chain», made of an extradossal chain with two additional inclined elements that connect it to the springers of the arch, an intradossal chain was added, an unused solution at that time because of its visual impact, but considered unavoidable in this case.

To evaluate, today, the structural behaviour of a vault we can, usually, refer only to the static equilibrium conditions. Assuming the deformability of the arch to be negligible and not resistant to tensile stress, a balanced solution may be found through the well-known graphic constructions. Using this approach, it is also possible to analyse complex vaults, reducing them as a composition of arches; the safe theorem assures us of the equilibrium of the actual structure. After checking the arch (or the vault), it is necessary to evaluate the support structures, in order to see if they are able to support the horizontal forces.

In the present case this approach is not sufficient, because the redundancy of the system imposes an analysis of the complete structure. It is not, in fact, possible to estimate, a-priori, the distribution of the horizontal force in the two chains, nor the contribution of the «sling chain», nor the deformation of the lateral walls.

We have, therefore, used a numerical approach through the definition of a finite element model. This method allows us to simulate, with sufficient accuracy, the different constructive details and the behaviour of the materials. In particular, we used the *fem* code ANSYS 5.5, in which the constitutive model for masonry developed by Gambarotta and Lagomarsino (1997) has been implemented, which permits the simulation of the progressive degradation of the stiffness (due to the cracking), as well as the actual tensile and compressive strengths and the presence of the friction, that limits the sliding in the mortar joints.

To study the system we have decided not to consider the 3D of the structure, both in relation to the constructive phases and to the shape of the structure, constituted by a number of parallel arches. From the building we have extracted a plane structural macro-system, made of the masonry arch, part of the vault, the chains and the lateral walls (modelled in the portion between two successive floors). The mediaeval wall, built on the keystone of the arch, is considered as a load, avoiding any eventual structural effect in the transversal direction.

The arch and the lateral walls have been modelled through a four-node element, in plane stress. The

chain system has been simulated through link elements, while a beam, which is linked to the masonry by gap elements, models the anchor. These guarantee a contact only in the case of compressive stress. The model is shown in figure 10, seen as solid by the attribution of different thicknesses to the different elements.

The material parameters are the following:

- *masonry*: $E = 2500\ MPa$ (elastic modulus); $\nu = 0.2$ (Poisson coefficient); $\rho = 2000$ Kg/m^3 (density); $\sigma_c = 4\ MPa$ (compressive strength); $\sigma_t = 0.15\ MPa$ (tensile strength);
- *iron*: $E = 210000\ MPa$ (elastic modulus); $\rho = 7850$ Kg/m^3 (density).

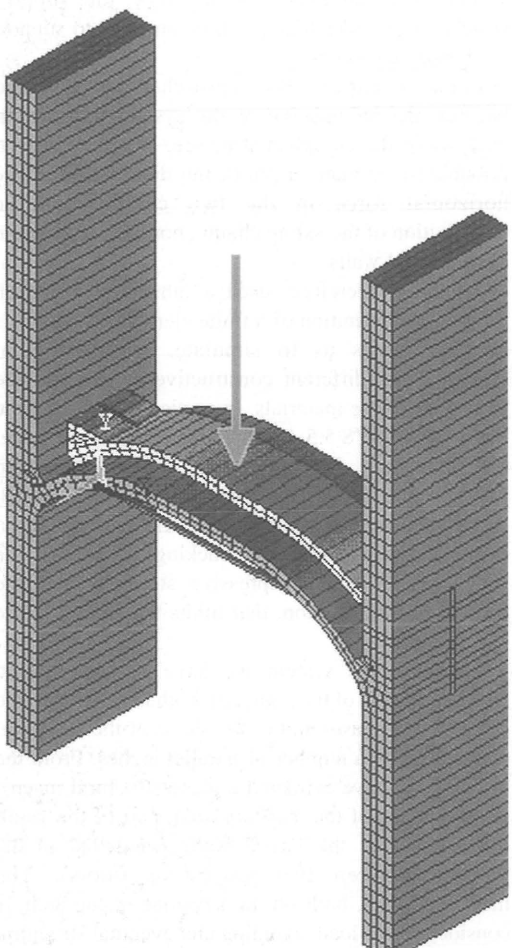

Figure 10
The finite element model

The fem model has been constrained at the ends of the wall, in correspondence with the other floors.

The individuation of the executive phases of the intervention, deduced by the stratigraphical analysis, suggests us to consider as load-bearing only the arch and the corresponding portion of the underlying vault, as we assume that the remaining part of the vault was built up after the removal of the propping structure. However, the notable thickness of the vault (25 cm) should indicate that in the design of the builder the entire vault works together with the arch, thanks to some trick in the propping sequence. Thus, two different models have been considered: A) the arch and the underlying portion of the vault; B) both the arch and the entire vault.

The first step of a finite element analysis of a complex structure is the evaluation of the elastic response, which doesn't consider the failure of the materials; this analysis gives a clear understanding of the behaviour for low actions and, usually, is enough for the operational conditions. In the presence of cracks, the linear analysis helps, anyhow, the interpretation of the instability, as it describes the situation just before the failure. The non linear analysis is, on the other hand, necessary to evaluate the safety of the structure with respect to the collapse, that is to know how much a given load may be increased before the total failure of the building and not only by considering local cracks. In this case both the analysis have been performed, as the load induced on the vault by the mediaeval wall is relevant.

In Table 1 the main results of the two models (A and B), by considering the linearity of the materials or the above mentioned non linear constitutive law, are showed; in particular they are: the largest values of the stresses in the crown of the arch, tensile stress at the intrados (σ_I) and compressive stress at the extrados (σ_{III}); the vertical displacement in the crown (u_y); the tensile stress in the two iron chains, the upper ($\sigma_{extrados}$) and the lower one ($\sigma_{intrados}$).

The results clearly show the relevance of the non linear analysis, for a correct interpretation of the structural behaviour. Indeed, in the linear analysis the tensile stress in the crown of the arch results

Table 1. Results of the finite element analyses

	σ_I [MPa]	σ_{III} [MPa]	u_y [mm]	$\sigma_{intrados}$ [MPa]	$\sigma_{extrados}$ [MPa]
Model A - (linear analysis)	*0.41*	*–0.95*	*2.4*	*19.3*	*15.9*
Model B - (linear analysis)	*0.25*	*–0.85*	*2.2*	*23.9*	*18.4*
Model A - (non-linear)	*0.12*	*–1.02*	*2.7*	*19.7*	*17.4*
Model B - (non-linear)	*0.14*	*–0.92*	*2.3*	*24.0*	*18.5*

inadmissible with the masonry strength; on the contrary, the non linear analysis considers a cracked zone, in which tensile stresses are significantly reduced, with a consequent increasing of the compressive stresses in the extrados; moreover, also the vertical displacement and the pull in the chains increase. Moreover, the analysis of the results shows the important role of the vault; indeed, if all the mediaeval wall weighs only on the arch, the tensile stresses would be too high to be sustained without wide cracks. Instead, if the constructive phases were such as to weigh also the entire vault, the structural system would result appropriate, as proved by the fact.

Finally, some considerations are addressed to the system of chains. The finite element analysis shows that both the intradossal and the extradossal chains work significantly, while the inclined chain are completely unloaded. It is worth noting that the slinging chain system (extradossal chain plus two inclined ones), used in order to hide the chains, has a doubtful effectiveness; this fact may be deduced both from structural analyses and by direct observations (in many cases the inclined chain appears to be inarticulate with the extradossal one, in correspondence of the connection). In this particular case, due to the difficulty of the intervention, the builder adopted a double security, by adding to the system also the intradossal chain; this fact made the inclined chains almost useless.

NOTES

In this paper, the structural analysis was written by Sergio Lagomarsino (DISEG, Università di Genova), the first four paragraphs were made by Anna Boato (DSA, Università di Genova), the remaining part was made by Daniela Pittaluga (DSA, Università di Genova). Drawings by A.Boato and M.Sarcina.

1. With reference in particular to an inter-university research project entitled «Costruzione voltate in muratura» («Masonry vault construction») undertaken in the years 1998–2000 in which Anna Boato and Daniela Pittaluga participated, in the context of the Genoa operative unit coordinated by Prof. Paolo B. Torsello (see Boato 2001; Pittaluga 2001).
2. The pavers were a little smaller and thinner than bricks of the same age.
3. Corresponding to standards of deviation that were relatively low.
4. A palm is equal to 24.776 cm (Rocca 1871).
5. For example, this can be seen in the following document, in which the choice whether or not to use a chain is dictated by the builder: *tutte queste stanze vanno in volta [di] mezza botte or a croxera senza ferri o con ferri bisognano ma ben secure et ben fatte*, «all these rooms are to be vaulted with barrel vaults or cross vaults without irons or with irons as need be but well secured and well made» (Archivio di Stato di Genova, Notai Antichi, 1840, 22 February 1549).
6. *. . . mettere ad ognuna di dette volte i telari de chiavi con suoi braccij di ferro da quattro a fascio con stanghette per longo e per traverso de manera che sijno molto forti e secure*, « . . . put on each vault the key frames with its iron branches four to a strip with small bars longwise and crosswise in such a way that they are strong and secure» (Archivio di Stato di Genova, Notai Antichi, 5137, 8 August 1629).
7. The church and its annexes, among which are the object of this study, correspond to number 34 in Piazza Santa Maria in Passione, which today is the property of the Università degli Studi di Genova.
8. The width of the space above various from approximately 3.15 to 4.0 meters; its total length is equal to 16.4 meters. The difference in length could be due to the fact that only the area that could be effectively used by the nuns was taken into consideration, to the exclusion therefore of the part that is the presbytery.

9. It has not been possible to verify the physical continuity of the anchor elements. However, in the only situation where there is almost complete visibility there has been several proofs of this. It can be seen above all in the two lengths of anchor element that are visible thanks to studies undertaken on the plaster that covers it. The section of this is equal (about 5 cm by 5 cm, and it remains constant for the whole part that is visible. This indicates naturally that it could be a single element, but it also constitutes an anomaly with respect to the wedge form that usually characterizes the Genoa anchor elements. Such an anomaly could however be explained precisely because the anchor element is a single element: its setting in place through several eyes, placed each a certain distance from the others, and its being placed under uniform stress are in fact rendered easier by a anchor element of a uniform section, aided by metal wedges placed separately in the several eyes (in fact, such wedges are indeed present).
10. The various pavements and rafters sat o the rubble; these were removed during the course of the restoration work.
11. The selection process could have been executed on the work site or could have been due to the acquisition of materials of a different quality.
12. The presence of lime gravel (or *crudi*) can be noted in many structures.
13. A correct interpretation of the data relative to the mortar must take into account the possible degradation that could have had a considerable influence on these characteristics.
14. These considerations are possible only if a large number of coeval structures are available for study.
15. For example, the analysis undertaken on 139 bricks related to the structure in question indicate a standard deviation of 0.27, a value that is relatively low for the seventeenth century. This could be related either to the extremely good quality of this lot of bricks or to a careful selection of the elements to be employed.
16. Medieval bricks, probably obtained from the partial demolition of the wall underneath which the vault was constructed, were found in the supports of the transversal arch and in the structure above the climbing arch that terminates the vault.
17. The observations were made in the visible areas: at the extrados near the center of the vault and at the intrados in correspondence to the last two transversal arches, which are also in the area at the center of the vault.
18. This data is also contrary to what is usually to be found in other structures of the same period. In fact, there is often an alignment of the joints in two (and, more rarely, or three or more) courses.
19. The vaults taken into consideration where those destined to be covered in plaster, as was the seventeenth century vault of Santa Maria delle Grazie.
20. The observations regard a portion of the vault examined at the intrados. Even though it was a small portion, it is fairly significant because, since it regards the area in the center of the vault, it allows the observations of one of a zone that is most characteristic and sometimes contains the greatest number of anomalies in the pattern.
21. One course is constituted entirely of brick headers (which means that on the side of the arch only a single brick can be seen); the next course is constituted of brick lengths (on the side of the arch there are two bricks that are visible). In the thickness of the arch there is the hook of the chain of the extrados with its *bracci* or arms: in correspondence to this several small anomalies are found in the brick pattern of the arches.
22. On the whole, the solid supports at the two extremes of the arch concern 2/5 of the width of the arch itself.
23. The elements of lesser thickness usually present a simple bevel at their edges, while those of a greater thickness generally have a «prismatic» profile.
24. At the intrados the large slabs of slate are not visible; the visibility is limited to only a few zones. It is, however, possible that the slabs, even if they do not concern the entire thickness of the vault, and in some case of the vault and the large arch above, are in any case such that they create a ring of at least 2/3 of the entire thickness.
25. The additional observation of the relationship between transversal arch and shallow connecting arch and, in the case of the end portion of the vault, of a tie of the longitudinal climbing arch to the transversal one, has permitted the reliable determination of the chronology of the construction campaigns.
26. These are two blocks of lime marly limestone of some 15 cm by 20 cm placed immediately above and below the wooden wedges and the small iron bars. The upper block presents noticeable signs of degradation; this should be releated to the way they were handled when they were «pushed» into place.
27. Unfortunately it was not possible to observe how this element was placed with respect to the transversal arch. In the lower part, in fact, where it touches the arch, the wood is extremely decayed. Further, work recently preformed block the visibility of the extrados of the arch precisely in correspondence to the hypothetical point of contact between the wood and the arch. The wooden mounting may be in relationship to a hidden chain that passes diagonally in front of it and dies within the arch. The role of such a chain has not yet been clarified.
28. The heads of these iron elements have average dimensions of 2 cm by 8 cm.
29. The wooden rafters have a circular section with an average diameter of 8 cm; they are placed approximately 70 cm from axis to axis. Wood chips are

to be found all around the rafters. Elements that are similar in shape, position and direction are found in the walls opposite them. The stratigraphic analysis of the portion of the adjacent masonry the rafters will be fundamental in the clarification their role in the underpinning operation, so that their context and later insertion can be determined. Other tests that could be performed are the dendrochronlogical examination of the rafters to determine their dating and an analysis of the mortar in contact with the wood.

30. To determine the limits of the construction campaign it would be necessary to perform a stratigraphic analysis on the lower portion of the longitudinal wall, which is, however, plastered and ornamented. A doubt remains about the dimensions of the cases of the largest arches: were they made at one time or at different times? A detailed analysis of one of these arches, and in particular on the mortar joints, could provide good results.
31. This date is provided by the memorial stone that is still in place in the end wall. The eighteenth-century style stucco decorations are therefore to be attributed to a later redecoration.
32. The dimensions of the brick can in fact be dated back to the first decades of the seventeenth century and the mortar turns out to be constituted of the sand of Sampierdarena.
33. This is understandable precisely when seen in relation to the daring of the campaign.
34. These chains turn out to be at the same height at which, in the space, are found the chains of the intrados.
35. The chains utilized are generally of a square section of approximately 5 cm per side. They present signs of hammering with a rounding of the corners. On the chains of the intrados it is possible to see the points where the single pieces used were welded. On average, the length of the bars is approximately 2.0 m.
36. This wall is plastered and covered with stucco decoration, so consequently it is unthinkable to perform any investigations. It has been inspected in correspondence to the right side, and on the portion that is visible there appears no sign of any anchor element, though the presence of chains cannot be excluded in the part that is not visible. It has not been possible at present to verify this with a Geiger counter.
37. This much has emerged from the analysis of what exists. The databank contains more than 300 structures and was realised by D.Pittaluga (with the collaboration of E. Calza, I. Chiappe, M.S arcina, P. Pittaluga, A. Canziani, L. Comino) with the assistance of G. Beltrame and F. Ciribì (curator of a databank at the Faculty of Architecture at the University of Genoa regarding research on vaulted structures that has been undertaken in the last ten years).

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Development of structural connections of steel truss bridges around 1900

Klaus Brandes

The development of structures and its details is determined by both structural requirements and the main stream of contemporary architectural art.

An interesting detail of such a development is the change of the constructional type of riveted connections of truss girder bridges in cities.

Within some decades, the form and the dimensions of the connections using so called gusset plates have changed considerably. That can be recognised by some bridges in Berlin starting from a simple form, after that changing to a more ambitious type caused by discussions about architectural aspects of subway viaducts in Berlin during erection of the first subway line and at the end coming to a simple technical type which is not satisfactory in terms of architectural quality, however, following the new ideas of Bauhaus and Functionalism and aspects of structural safety.

The development is presented by giving some examples and mentioning the discussion at that time.

An engineering aspect is introduced by looking at the structural behaviour under severe loading and explaining «weak» structural details which had to be changed for the design of future bridges.

Some research work performed recently at BAM, Berlin, opens the opportunity of rational investigations and rating of those bridges for extending their service life considerably, sometimes with some strengthening and additional inspection procedures.

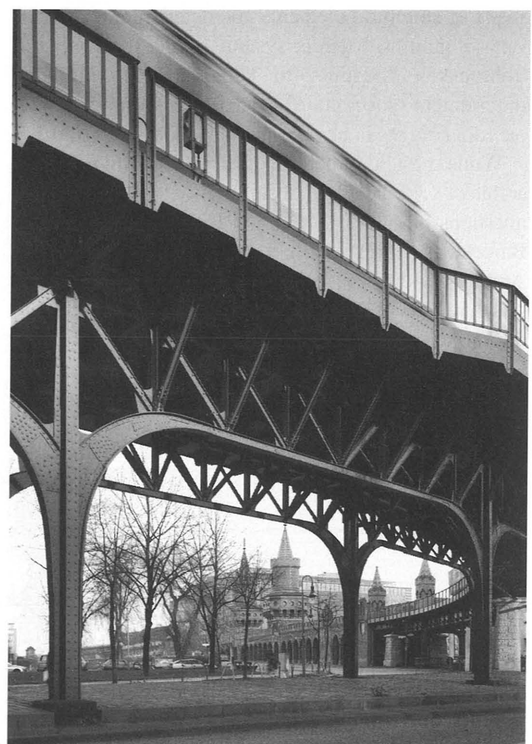

Figure 1
Viaduct of the line 1 of the subway in Berlin-Kreuzberg, completed 1902, a filigran truss girder system

INTRODUCTION

An important and interesting part of architecture is «Engineering Architecture» which not necessarily is the outcome of architectural design, rather then of engineering thinking and insight. Nevertheless, this manner of engineering design is influenced by the stream of contemporary arts. Examples for Engineering Architecture are numerous, e. g. the Eiffel Tower, Paris and the CN Tower, Toronto, and bridges like the Golden Gate Bridge, San Francisco, and the Tower Bridge, London.

A less ambitious structure is the steel viaduct of the subway system in Berlin which had been completed in 1902 —exactly hundred years ago. Studying the viaduct, the combination of optimal design in terms of economy and safety and in terms of esthetics becomes obvious (Landsberg 1904), figure 1.

The forms of Art Nouveau (called Jugendstil in Germany) have influenced the design with the pleasant shape of structural elements and details. However, the very filigran truss girder system —giving a feeling of lightness— conflicts to some extend with the requirement of structural safety. The contrast to later on built bridges is obvious when looking at figure 2.

While most buildings will be designed only following architect's ideas about forms which do not concern questions of statics —e. g. the Buildings by Gaudi in Barcelona— bridges have to be safe in its first priority. And this requirement causes conflicts in many cases.

There are several examples of structures which outline the paths of forces like the Eiffel Tower or the Olympic Roof at Munich. Their form is satisfying also in terms of esthetics, however, sometimes it is only interesting from an engineer's point of view like the new railway bridge in Berlin-Spandau, figure 3.

Truss girders were an outcome of looking for stable bridge structures which minimise the weight. Their overall design as well as their structural details are subject of looking for adequate appearance.

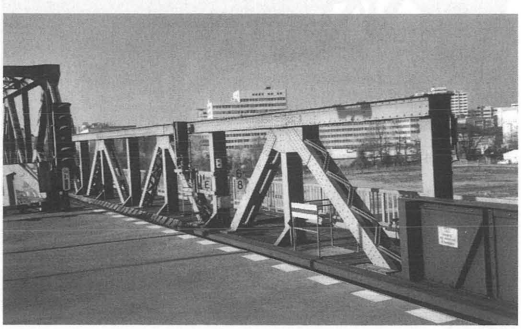

Figure 2
Bridge of the Berlin subway built some years later on

Figure 3
A railway bridge in Berlin-Spandau built in 1997. The heigth of the girder follows the stressing of the girder

STEEL VIADUCT OF THE SUBWAY BERLIN

The «Hoch- und Untergrundbahn» (subway) Berlin opened its service in 1902. The system has been

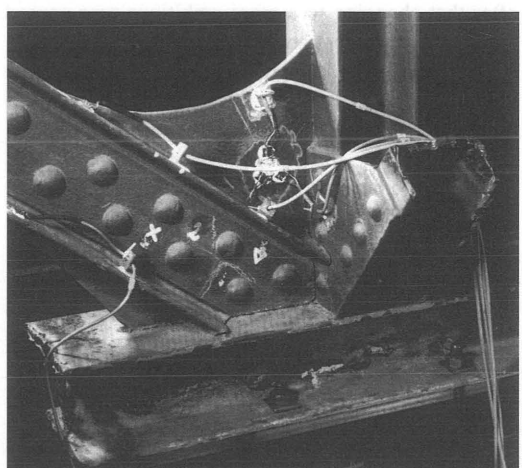

Figure 4
A node of the viaduct

extended during the following years. Some of the original parts have survived war time and demolition mania later on. Now they are protected by law as monuments of Engineering Art (Jäger und Wachter 1999).

The figures 4 and 5 give some impression about the development of the systems and the details.

Considering the viaduct at Berlin-Kreuzberg, the optimisation of the gusset plates can be studied. Even now, we are able to analyse the stressing of different structural elements, also —to some extend— the stresses in the gusset plates and their response to fatigue loading —the ever and ever repeated loading by traffic which— at the end —causes fatigue cracks and failure (Brandes 2000).

As far as we could find out by modern engineering approaches and by many fatigue testing on complete original bridges (Helmerich und Brandes 2002), the engineers who designed the viaducts have succeeded in constructing very small gusset plates, however reaching a stress level at the most stressed regions that caused doubts about the resistance to fatigue loading for 100 years or even more.

After extensive investigations, we found, that the real stress level permits a life time for more than hundred years, may be two hundred years. Nevertheless, in some other cases, bridges of a similar type had to be replaced after only some years because of cracks in the gusset plates.

THE MECHANICAL EVALUATION

The investigation, evaluation and rating of existing bridges is more and more common, mainly stimulated by the lack of financial means of the owners of bridges. In most of the cases, hidden reserves of load carrying capacity can be discovered. That means that the bridge can be used for some more years or decades, sometimes with some minor repair or strengthening.

For engineers, the investigation of an existing bridge is quite different to the straight forward procedure of designing a new bridge according to today standards. After many decades of service, bridge elements may have suffered from loading or the attacks from the environment. That has to be included in a safety assessment, however, no approved procedure is available how to do this in a satisfactory manner.

It has been a very interesting challenge to find out the real behaviour of the viaducts by measurement of the forces within the structural elements under traffic loading (Herter, Fischer und Brandes 2002) and by testing some dismantled bridges of the same kind which had been replaced before, in the big testing hall of BAM (Helmerich und Brandes 2002), figure 6.

During the tests in the testing hall, we took the opportunity to search for hidden cracks in the gusset plates by radiographic inspection, figure 7. This task is very complicated because the cracks always start from rivet holes and are covered by some steel layers

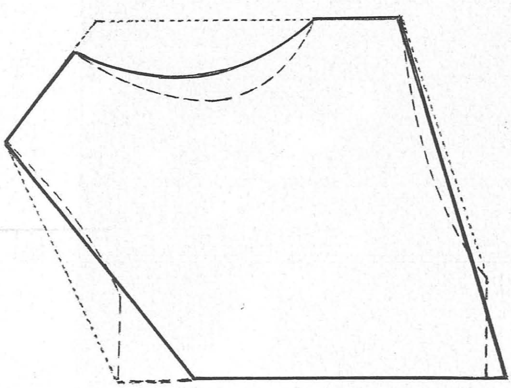

Figure 5
Development of gusset plates' form : Full line: real system; small dotted line: unfavourable in terms of appearance; dotted line: stressing of the gusset plate becomes to large

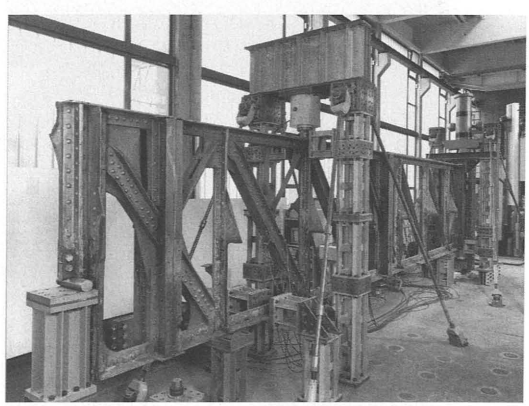

Figure 6
Fatigue test on an original truss girder of the subway Berlin in the testing hall of BAM

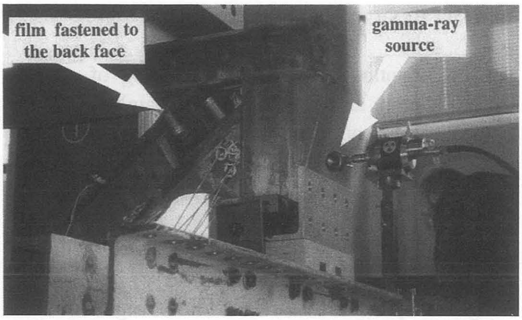

Figure 7
Radiographic inspection during tests on subway truss girders at BAM

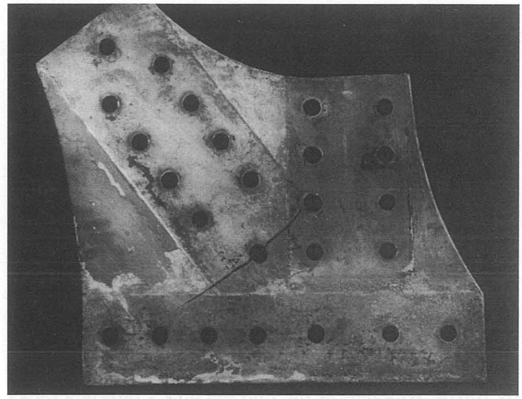

Figure 8
Crack in a gusset plate after millions of loading cycles

and the heads of the respective rivet. The result of the tests was very encouraging. We found that cracks can be detected when they reach about 4 mm length. Then, the future propagation of cracks can be assessed. In most cases, many years are needed to come to a crack length which may cause danger for the bridge, figure 8, figure 9.

While we performed the tests on a load level more then double of the real load, in reality, the crack propagation —if any crack will be initiated— will be very slow.

The overall result of the investigations is that the engineers who designed the viaduct had found the optimum of dimensioning the gusset plates. They are the weakest elements of the viaducts as the investigation showed, however, they are strong enough to resist the loading for a long period of time.

It should be mentioned, that some years after the construction of the viaducts in question, gusset plates have been dimensioned much thicker and larger. The

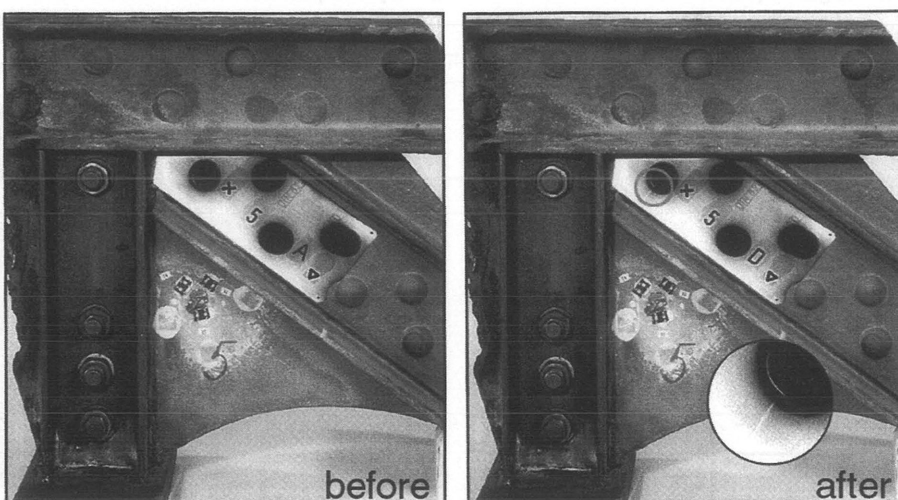

Figure 9
A detected crack on an X-ray film

reason for this change was that gusset plates shall not be the weakest structural elements because the inspection of gusset plates is nearly impossible. However, if other structural members are weaker, they exhibit first cracks which can be detected easily by routine inspection, and this is crucial for structural safety.

Figure 10
Stubenrauchbrücke in Berlin crossing river Spree

How to protect old steel bridges from being replaced ?

The recently developed methods of investigating steel bridges which are about 100 years old, offer a rational tool to assess the safety of the bridge sufficiently.

During the last decades, many old steel bridges have been replaced by new bridges because the knowledge about the condition of the bridges had not been satisfactory. Only improvement of the methods available for the evaluation of old structures has lead to a better situation. Just now, it should be impossible to demolish an old structure without applying all the new tools of *minimal invasive methods* for a sound rating.

Minimal invasive methods comprise all the methods which give a better understanding of the behaviour and the state of condition of a structure: Exact visual inspection, measurement of geometric conditions and strains and deformation under loading, non-destructive testing like ultrasound and radiographic inspection, dynamic identification of the structure etc.

The idea of minimal invasive techniques originates from the medical sciences where exactly the same methods are used to obtain a comprehensive view of the conditions of a patient.

There is much more to develop to avoid that monuments will be demolished only because of lack of knowledge about their structural safety. However, a first important step has been done.

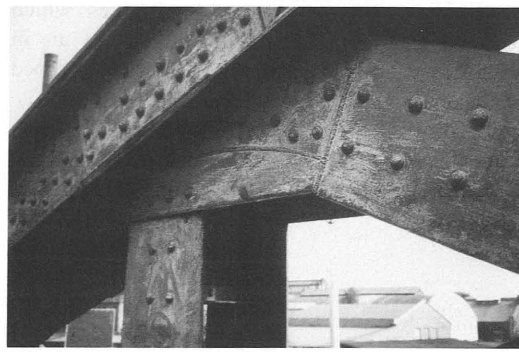

Figure 11
Strengthened gusset plate of the Stubenrauchbrücke

Another Example

A very important bridge in terms of history of engineering is the Stubenrauch-brücke in Berlin which was built in 1908, designed by the famous engineer Karl Bernhard (Bernhard 1908), figure 10. About 15 years later on, the bridge has been strengthened and at a second time in the 1930ths. The second strengthening concerned the gusset plates which had to be enlarged when following the new ideas of engineers from that time. Also the bars between the two arches were strengthened by adding broad steel plates which changed the appearance to the worse. In figure 11, the type of strengthening is clearly visible. The gusset plates had been enlarged by additional steel sheets which were welded to the existing plates. At that time, welding of the old material was not favourable, as we know now. However, when investigating the bridge in 1996, we could not find cracks near the weld seams.

With our newly developed tools, presumably, an investigation as that in the 30^{th} had had resulted in not doing any strengthening.

SUMMARY

The design of engineering structures like bridges referred in its forms to those of contemporary art. In performing an actual design, the requirements of structural safety sometimes conflicted —and conflict today— with ideas about forms adequate to the main stream of architecture of the time in question. It is of interest for understanding of the engineering art to study the development of structural systems and detailing regarding mechanical structural analysis on the one hand and favoured architectural forms on the other hand.

The shape of gusset plates is a very special case in that field of construction history.

Remark: It should be mentioned that for the preservation of monuments like bridges which underlie safety requirements as long as they are in use, methods of investigation as we have developed are indispensable.

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The Framework Truss: Development and structural analysis of framework trusses in the USA at the beginning of the 19th century

Tim Brengelmann
Rainer Barthel

THE DIRECTLY EUROPEAN PREDECESSORS

When you look at the European bridge construction context of the later part of the eighteenth century as it might have influenced American bridge builders, you are left with the impression that American builders did surprisingly little borrowing for their own development from the Old World for their structural ideas, despite the fact that some European wooden bridges were well known.

The significant difference was the lacking of a structural design clarity of the most European bridge structures, which were burdened with complicated, wood-wasteful carpentry, involving multiple Queen-post trusses within a given bridge, resulting in many diagonal members between the vertical posts, and with the most dazzling, labourintensive use of continuous zig-zag joints, secured with wedges, to laminate several timbers into a larger chord member.

The bridges of the Grubenmann family in Switzerland were internationally well known in its own time. Like the Schaffhausen bridge, Figure 1, the Wettingen bridge, Figure 2, was very famous and was frequently visited by interested travelers, resulting in a variety of written descriptions, dimensions and details. So if any European bridge can be said to have influenced an American bridge, it was probably the Wettingen bridge. It is a multiple Queen-post truss with a very high and stiff arch, consisting of multiple members of wood, fastend on each side to the abutments.

The further development in European wide span structure was determined to perfect both the wooden and the stone arch and later on, at the end of the eighteenth century, the arch made of cast-iron, particularly in France, the Pont d' Austerlitz, 1802–05, and in England, the Sunderland bridge, 1796.

So the first wide span structures in the USA started even with the arch as the substaining member of long-span wooden bridges.

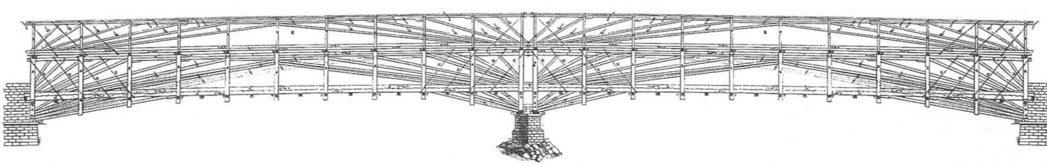


Figure 1
Grubenmann bridge at Schaffhausen, 1756–58, two different spans about 60 m and 53 m

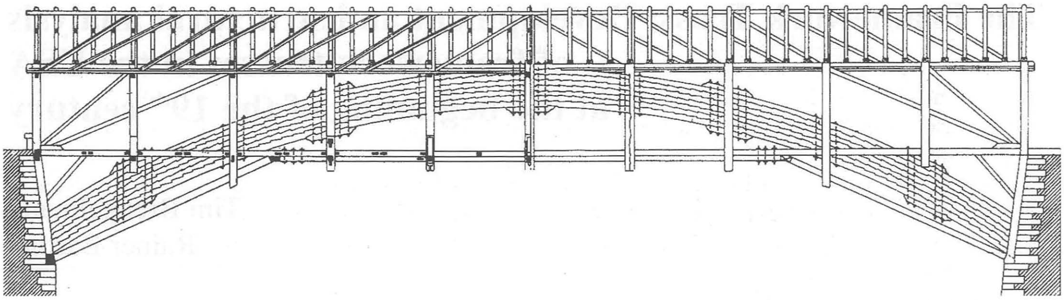


Figure 2
Grubenmann bridge at Wettingen, 1764–1766, free span about 62 m

Overview about the Developments in Building Framework Trusses in the USA

In 1804–05 Trenton Bridge, over Delaware, was planned and built by Theodore Burr. The superstructure consisted of five laminated wooden arches with the longest span about 60 m in the clear. The following bridges of Theodore Burr commonly used an arch truss. The truss is of the multiple kingpost type with a diagonal timber brace, sloping up towards the center of the bridge, between each panel post.

The multiple kingpost truss is among the most famous trusses and wide spread over the country. Built by local men in many modifications, variations, and reinforcements of the basic designs. They were used for spans of 15 m to 35 m.

1820 Ithiel Town designed and patented a covered bridge truss which could be quickly built by any good carpenter. The Town Truss consisted of a web of light planks criss-crossed at an angle of 45° to 60° like a lattice and fastened together with wooden pins or trunnels at each intersection. It was light and cheap and could be assembled in a few days' time. This design was well received and soon became most popular both for highways and later for railroads.

Colonel Stephen H. Long patented two types of trusses, the first in 1829 and the second in 1839, a further development and improvement of the first patent. The truss was essentially a multiple kingpost with counterbraces. The patent showed two single diagonals, but practically all bridges were built with two braces and one counterbrace in each panel, and with either one or two post.

The Warren Truss is a well known steel bridge design, that was initially buit in wood. James Warren and T. W. Morzani patented this simple light-weight truss in 1838. There are two different types of the Warren Truss, the single and double system. The single type consists of series of diagonal timbers placed in the form of a W with no rods or panel posts. The double system has a second similar set of diagonals which intersect the first.

The 1839 patented Haupt Truss shows a panel-type truss using single-latticed diagonal braces sloping up

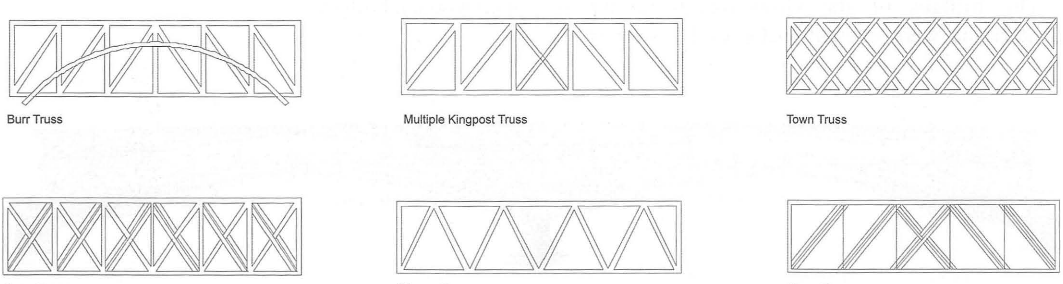


Figure 3: Types of framework trusses

towards the center of the bridge, each spanning three short panels and also braced with a full-length kingpost, serving in place of an arch.

Squire Whipple was the first man who built a diagonal braced steel bowstring arch in the USA in 1840. The arch itself was made of cast iron and the tension members were carried out in steel. Whipple was even the first to publisch theoretical investigations about framework-trusses. He started his investigation of a framework at the abutment and then he gradually calculated the forces of every single member. He solved this problem also by using the polygon of forces.

In 1840, William Howe, conceived and patented a truss similar to the Long but with a most important improvement. He substituted iron rods for the wooden post as tension members, eliminating one heavy timber and providing a means of easy adjustment by having screw ends with washers and nuts. The rods could be easily shipped and the truss timbers prefabricated. The Howe Truss gradually replaced other trusses and it became the most popular design during the last half of the nineteenth century (Wilson).

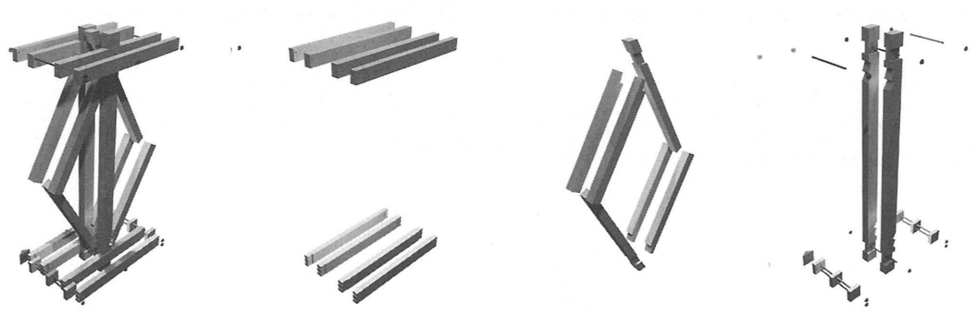
Reconstruction and Structural Analysis of some important stages in early US Framework Design

Structural Analysis

The structural analysis of different bridge types should inform about the structural behavior of each framework system and its used connections. In the first step the connections between the members were approved as unplibly hinged together and in a further calculation the connections were looked upon to be pliable and they were described in the statical model as linear-elastic springs. Thereby the different framework systems could be compared to each other by their structural behavior and you can give an opinion of the reaction influenced by the connections to the whole system. As further step the behavior of the framework was examined for asymmetrical loading. Thus a uniform load of 5.0 kN/m was applied on half of the bridge length.

The Long Truss

The relevant way of joining the members of the bridge represented in Colonel Longs patents was still the treenail although steel connections already had a far spreading. Main changes of the connections, not concerning to their material, referred the improvement in its second patent but primarily to the arrangement of the connection of the posts, chords, braces and counterbraces among themselves. To be able to readjust developed lowerings wedges were imported to the significant places. The basis of the computation became the truss of the Brownsville Covered Bridge from the year 1840 (Historic American Engineering Record). It was selected, because this bridge is documented exactly in it dimensions and detail remarks. The light span of the wooden bridge amounts to about 44,80 m with a system height of 5.10 m. The upper, like lower chords consist of two 30 × 14 cm and two 30 × 16,5 cm strong timber beams. In each panel there are two posts measuring 30 × 20 cm. The diagonal struts are



A single panel of the Long Truss Upper and lower chords Diagonal struts Vertical posts

Figure 4

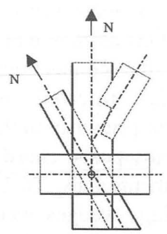
Redevelopement of the Long Truss

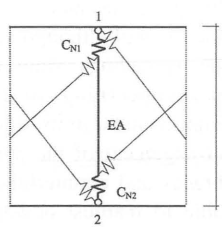
implemented and by means of a front disalignment with the posts as two bars 23 × 20 cm. Circulating steel strips secure the situation of the connection. A 20 × 20 cm timber beam is serving as diagonal tie. The connection of the chords, the panel posts and the diagonal tie becomes secured on the one hand over a 2 cm strong steel bolt and on the other hand over a releasing of the panel posts as well as the diagonal tie with the chords.

The connection of the members of the Long Truss was idealized separately for the connection of the steel bolt as well as the connection of the front disalignment in the computer model:

Pliable connection of the posts and tie beams
(Informationsdienst Holz)

Connection		Static Model	
Pliability	v	Change of Length	Δs
Displacement Stiffness	C_N		



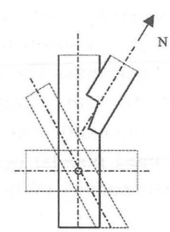


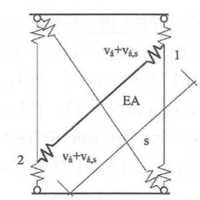
$v_N = [N/C_N]$
$= v_{N\,R,d} \cdot [N/N_{R,d}]$
$C_N = N_{R,d}/V_{N\,R,d}$

$\Delta s = [N \cdot s/E \cdot A] +$
$+ [N/C_{N1}] + [N/C_{N2}]$

Pliable connection of the diagonal struts
(Informationsdienst Holz)

Connection		Static Model	
Pliability	v	Change of Length	Δs





$v_\alpha \approx 1{,}5$ mm
$v_{\alpha,s} = \Delta u \cdot \alpha_s \cdot h_s \cdot \sin\alpha$

$\Delta s = [N \cdot s/E \cdot A] +$
$+ \Sigma[v_\alpha + v_{\alpha,s}]$

The following graph represents the deformations of the Long Truss under self load with and without consideration of the pliable connections on the one hand and on the other hand the additional influence of a half-sided load:

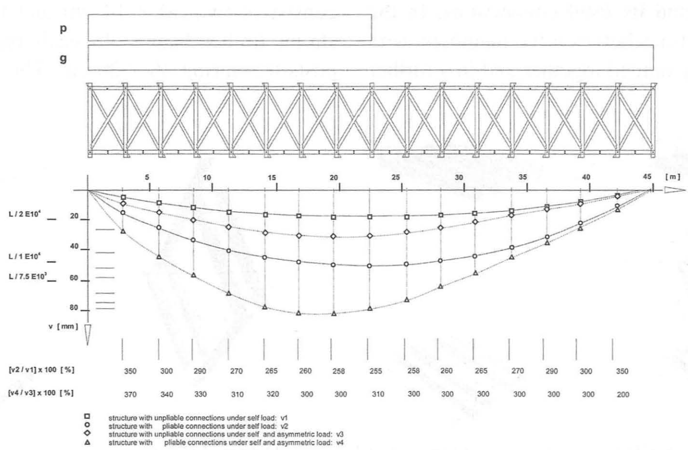


Figure 5
Displacements of the Long Truss

Conclusion Long Truss

The consideration of the pliable connections leads in the case of the Long Truss to a substantial deformation increase, which amounts to 255 % deformation surname in the center of the truss. Thus one receives a span / deformation relationship of L/1500 for the load case self load. The system of the Long framework reacts under the influence of an asymmetrical load by the presence of the counterbraces to a balanced deformation figure.

THE HOWE TRUSS

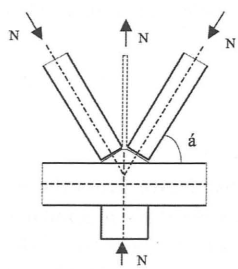
Howe completely maintained the system of Long in view to the structural arrangement of the structural elements. He did not take a construction member away added, also none, but only the wooden posts, which are designed as suspenders, were replaced by steel tension bars. He solves the connection of these tension bars with the braces, counterbraces and chords by a counter bearing made from oakwood, this detail was later substituted as a cast-iron shoe whatever in the most diverse forms.

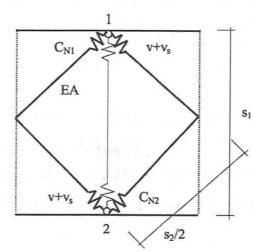
For the computation of the Howe Truss the model is based on a bridge represented in a report on a journey through North America published by K. Culmann in 1851 «Der Bau der hölzernen Brücken in den Vereinigten Staaten von Nordamerika» (Culmann). The light span of the wooden bridge amounts to about 32,00 m with a system height of 3,40 m. The upper chords consists of three 25 × 22 cm strong timber beams, in different to the lower chords which consists of four 25 × 16 cm beams. The diagonal struts and ties have both the dimension of 23 × 16 cm, but the diagonal struts are built with two braces in each panel. Two 4,5 cm traction ties are used as vertical rods instead of common wooden posts. They are fixed at the end with cast-iron washers against wooden blocking elements. A cast iron shoe is used to connect the chords, braces and counterbraces.

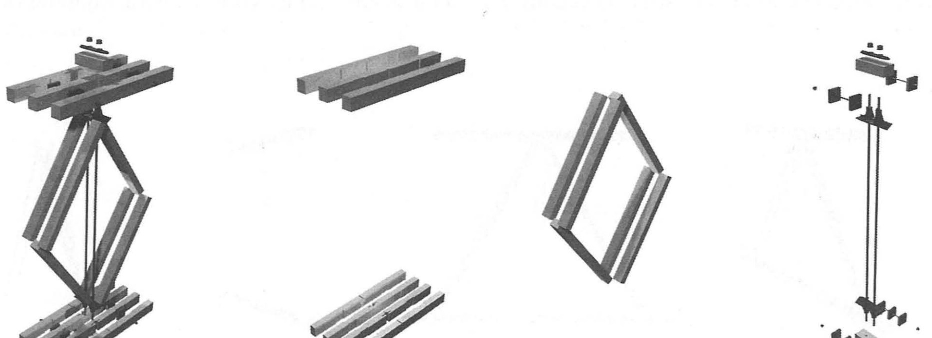
The traction forces that vertically rods and the thrust forces from the diagonals cause pressing transverse to the wood fiber in the chords. The resulting deformations are considered as spring elements in the computer model:

Pliable connection of the diagonal struts
(Informationsdienst Holz)

Connection		Static Model	
Pliability	v	Change of Length	Δs







A single panel of the Howe Truss — Upper and lower chords — Diagonal struts — Vertical rods

Figure 6: Redevelopement of the Howe Truss

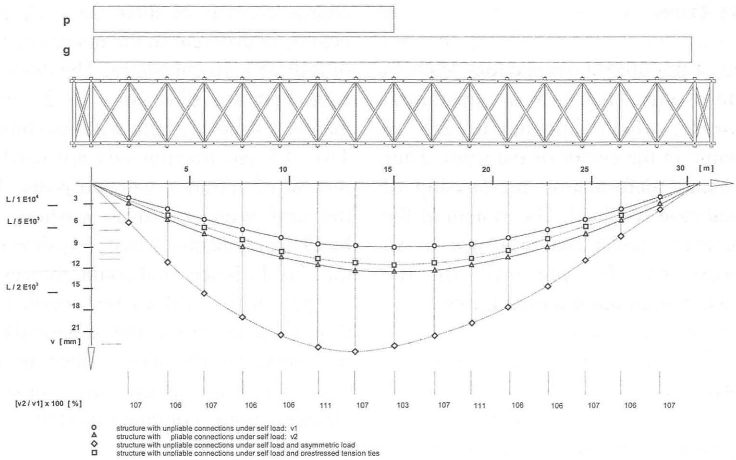


Figure 7
Displacements of the Howe Truss

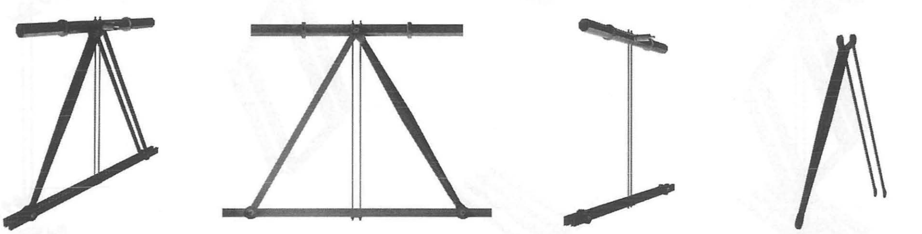
The graph represents the deformations of the Howe Truss under dead load with and without consideration of the pliable connections on the one hand and on the other hand the additional influence of a half-sided load (Figure 7)

Conclusion Howe Truss

The deformation increase of the Howe Truss due to the consideration of pliable connections precipitates relatively small. This is in particular because of the fact that the intelligent construction of the Howe Truss works without tension-stressed connections. By a pre-stressing the vertical rods around 1mm/m the deformation of the system can be reduced more under dead load even by 20 %. The truss receives a span / deformation relationship of L/2500 for the load case dead load. The Howe system behaves also durably in relation to asymmetrical load, which the even process of the deformation figure shows.

THE WARREN TRUSS

The Trent bridge, 1851 delighted by Cubitt after the system Warren, represents the first iron bridge in a consistent frame-work construction(Dietrich and



A single panel of the Warren Truss | Side elevation | Upper and lower chords | Diagonal struts

Figure 8
Redevelopement of the Warren Truss

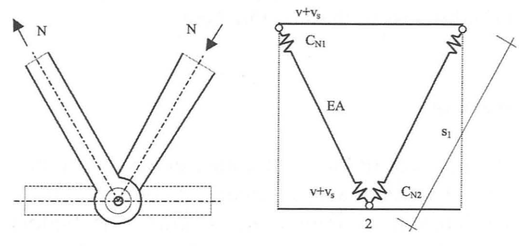
Heinzerling). Under uniformly-distributed load all construction units are trained demand-fairly in form and material, the struts consist continuous of cast irons members and accordingly the tension members are manufactured from iron. The upper chord is poured as cast-iron tube, outside with an octagonal cross- section and inside with a circular cross-section, with a diameter of about 12 cm and a minimum wall thickness of about 3 cm. The diagonally struts are cast-iron hinged columns with a cross-shaped double-T cross section, which becomes broader the center of the column. The lower chords and the diagonally ties are both tension ties with rectangle cross- sections. There are two diagonally ties each panel measuring about 18 × 2 cm. The lower chords could be assembled accordingly to the tension and measured about 22 × 2 cm each. All members were connected by iron bolts, about 3 cm in diameter. The light span of the Trent bridge amounts to about 72,00 m with a system height of 4,70 m. (Figura 8)

The pliable connections of the Warren Truss are principally based on the bending of the bolts caused by the eccentric connection among each other

The graph represents the deformations of the Warren Truss under self load with and without consideration of the pliable connections on the one hand and on the other hand the additional influence of a half-sided load (Figure 9).

Pliable connection of the diagonal struts and ties (Informationsdienst Holz)

Connection		Static Model	
Pliability	v_N	Change of Length	Δs



$$v_N = [N/C_N] = v_{N\,R,d} \cdot [N/N_{R,d}]$$
$$C_N = N_{R,d}/V_{N\,R,d}$$

$$\Delta s = [N \cdot s/E \cdot A] + [N/C_{N1}] + [N/C_{N2}]$$

Conclusion Warren Truss

The Warren Truss shows a uniform increase of the deformations due to the consideration of the flexible connections at a value of 106% in each panel. The span / deformation relationship in case of self load is

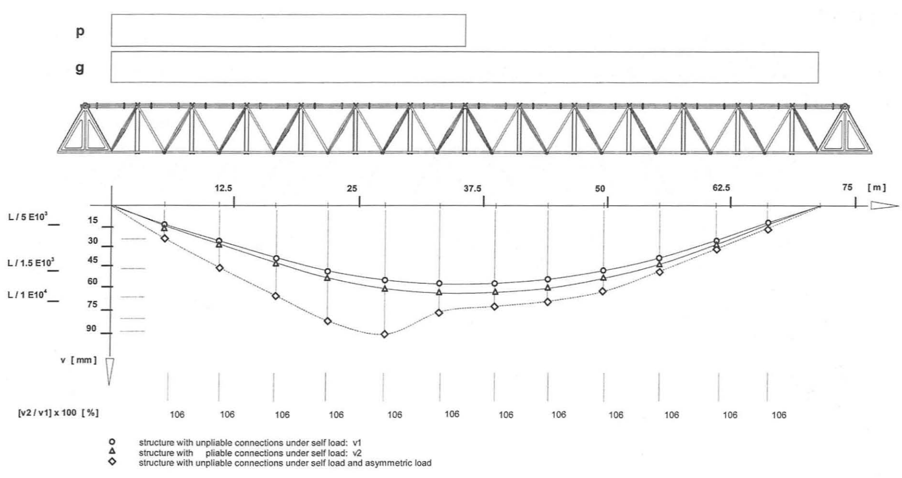


Figure 9
Displacements of the Warren Truss

about L/1150. Although the Warren Truss shown here is trained outstanding according to the framework theory for the case of a uniformly-distributed load, it reacts very sensitively to an asymmetrical load situation. This is above all of the fact that pressure-forces arise in the tension bars under an asymmetrical load within certain ranges of the truss itself. This leads to a loss of the diagonals concerned and thus to a rigidity loss of the structure here.

Summary

The first half of the 19th Century draws out in three-ways regarded as particularly important for the development of framework systems. The building material wood is replaced gradually by the more efficient material steel. The system formations achieve a variety at structure combinations, never known, to obtain larger spans and higher load-bearing capacities and the theoretical bases for the calculation of a framework were found.

The comparison of the three selected framework systems shows the efficiency of the building material steel as a basic element and by the example of the Howe Truss, although it is importantly shorter in its span however the same system formation is used as the Long Truss, shows itself already clearly its outstanding constructional training by the displacement behavior.

Consulting the influence of the used connections on the load-bearing behavior under dead load for the evaluation, the chronological classification of the systems is to be equated to the way the different framework systems react on their connections. Accordingly the Warren Truss behaves most durably to the resulting deformations in the used connections.

By the consistent employment of steel and cast iron in the Warren Truss it has been possible for the first time to reduce the necessary construction height strongly compared with the timber construction methods.However if the weight of the construction is the relevant evaluation parameter, then the Howe Truss obtains a outstanding result due to its extremely intelligent construction. However the question remains openly whether an appropriate Howe Truss for example of double span could be judged just as favorably.

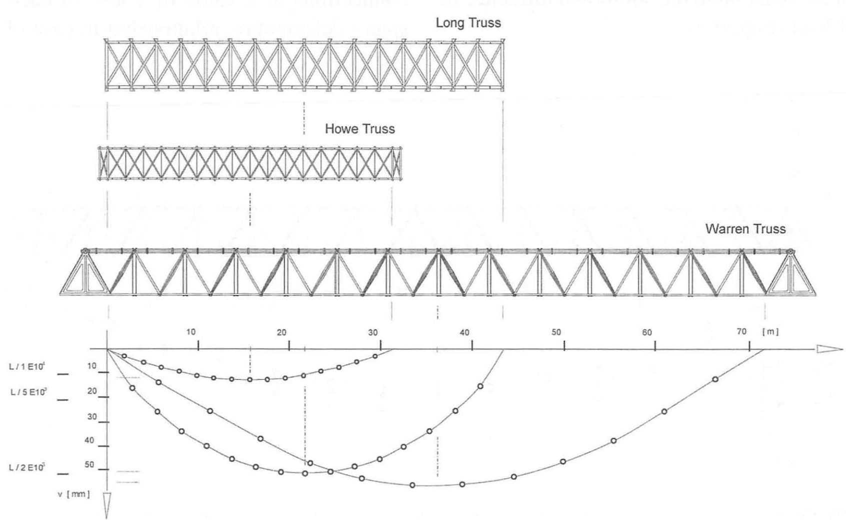


Figure 10
Representation to scale and comparison of the displacement behavior

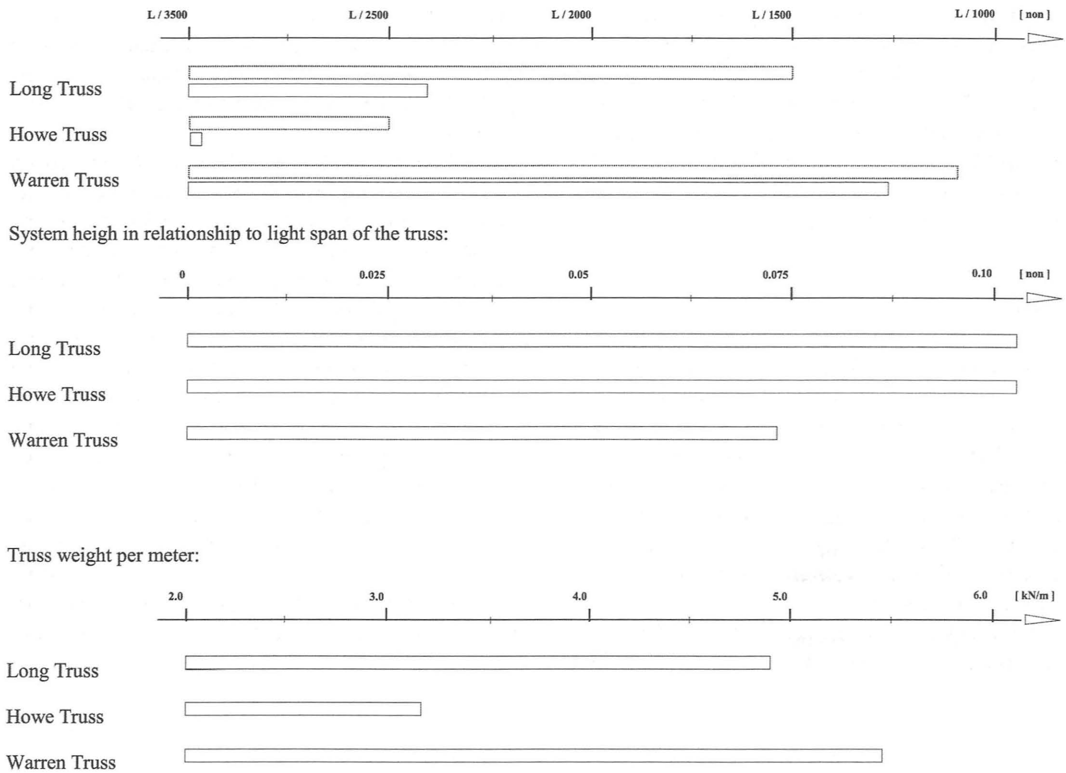


Figure 11
Span / deformation relationship with pliable and non pliable connections:

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Engineering analysis as a historical documentation tool: Recent work of the Historic American Engineering Record

Stephen G. Buonopane
Justin M. Spivey
Dario A. Gasparini

During the past decade the Historic American Engineering Record (HAER) has emphasized engineering analysis as a counterpart to its more traditional documentation methods of historical narratives, large-format photography, and measured drawings. This paper explores the use of modern engineering analysis as a historical documentation tool, based on examples from three bridges (Figure 1): Upper Bridge at Slate Run, Pennsylvania (1890), Lower Bridge at English Center, Pennsylvania (1891), and Bluff Dale Bridge in Erath County, Texas (1890).

For many of the 19th-century bridges that HAER documents, little or no technical information survives regarding their engineering design—the only resources are the bridges themselves and knowledge of bridge design methods of the day. Empirical rules or approximate calculations formed the basis for the design of most 19th-century bridges. All three bridges considered here combine multiple structural systems, the exact analysis of which was not possible at the time of their construction. However, modern structural analysis can reveal the actual load-carrying mechanisms of the bridges and can assess the accuracy of approximate design methods. Engineering analysis may also provide insight into otherwise unexplained features of a bridge. Unusual structural systems often encountered in the documentation of historic bridges can lead to a wider study of related structural forms.

Techniques of engineering analysis for historic bridges Analysis of statically indeterminate structures

All three of the bridges discussed here are statically indeterminate structures—loads are carried through multiple paths in the structure and the member forces cannot be calculated from equations of equilibrium alone. Accurate analysis of indeterminate structures requires consideration of deformations and solution of a system of simultaneous algebraic equations. Charlton (1982) attributes the first accurate analysis of indeterminate trusses to Claude Navier, published in his *Leçons* of 1826. Otto Mohr's methods, published in 1874, widely influenced engineers, although several other prominent theoreticians had contributed significant results in the intervening years. By the early 20th century, engineering textbooks regularly included analysis methods for indeterminate structures, including trusses, continuous beams, and suspension bridges (Johnson et al., Part II [1893] 1911; Merriman and Jacoby, Parts III and IV [1898] 1907). Even with theoretical grounding, analysis of highly indeterminate structures requires the simultaneous solution of a large system of equations, not practical for late 19th century bridge designers. Nevertheless, engineers often preferred indeterminate truss designs for their redundancy and other practical advantages. Many of the common 19th century truss types are, in fact, statically indeterminate. In contrast, the prominent American railroad engineer

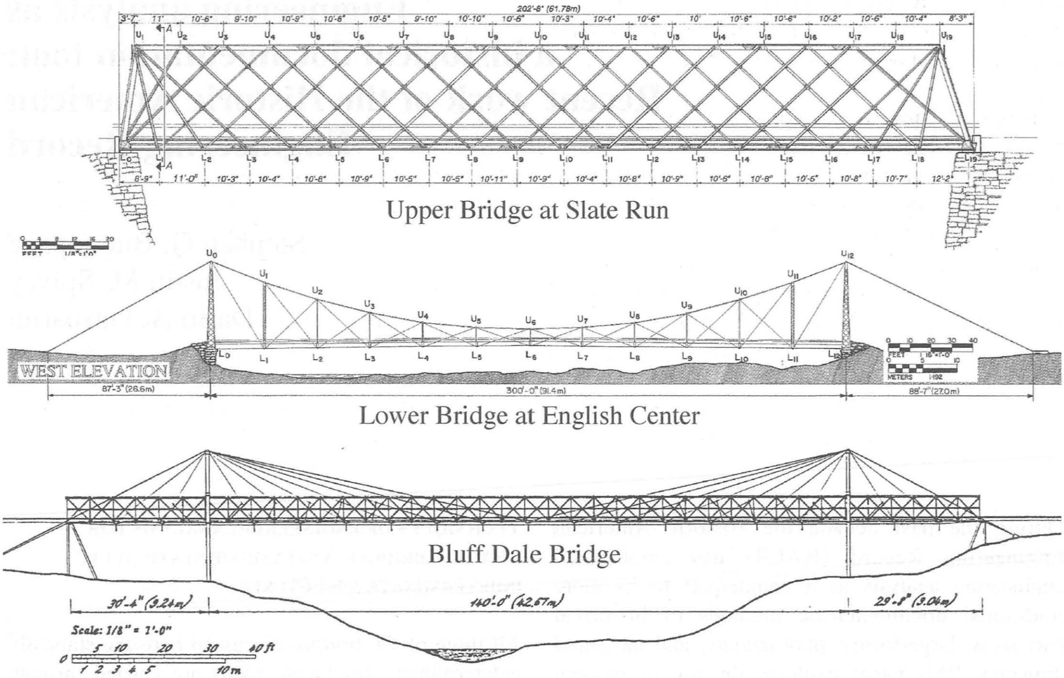

Figure 1
Elevations of the three bridges documented by HAER incorporating engineering analysis (HAER PA-460, Elizabeth Milnarik and Slavica Bubic, delineators; PA-461, Michael Falser and Jonathan Cherry, delineators; TX-36, Zsolt Zsanda, delineator)

Theodore Cooper promoted pin-jointed, determinate trusses in large part because they could be analyzed without resorting to approximate methods (Gasparini and Simmons 1997).

Modern structural analysis of indeterminate structures is performed using the finite element (FE) method, in which the structure is subdivided into many discrete elements each satisfying basic principles of equilibrium and deformation compatibility. A computer program assembles and solves the large system of equations, and calculates forces and displacements of each member.

Analysis of geometrically non-linear structures

Cable-supported structures commonly exhibit significant shape change under applied loads, known as geometrically non-linear behavior. Calculation of the deformations and internal forces of cable-supported structures requires a mathematically sophisticated analysis because the equilibrium must be enforced on the deformed configuration of the structure. The development of the parabolic cable suspension bridge in the early 19th century led Claude Navier to publish the first non-linear analysis of the unstiffened suspension bridge in his *Rapport . . . et memoire sur les ponts suspendus* of 1823. Josef Melan extended the non-linear theory to suspension bridges with stiffening trusses by 1888, but his theory was not employed in the design of an actual bridge until the Manhattan Bridge of 1909 (Buonopane and Billington 1993). While stiffened suspension bridges are only a few degrees statically indeterminate, cable-stayed bridges are many times indeterminate as well as geometrically non-linear, and thus could not be accurately analyzed in the 19th century. Modern

structural analysis of geometrically non-linear structures can be performed with incremental FE analysis or numerical solution of the non-linear equations.

HAER documented a group of cable-supported bridges in rural Texas, including parabolic cable suspension bridges, cable-stayed bridges, and suspension bridges with inclined stays (HAER TX-104). Proper structural analysis of these bridges required the use of geometrically non-linear analysis—both non-dimensional analytical models to understand the fundamental influences on the behavior and detailed FE models to calculate member stresses.

Evaluation of approximate design methods

Engineering textbooks such as Johnson et al. (Part I [1893] 1914) or Merriman and Jacoby (Part I [1888] 1922) provide design methods for bridges of common structural forms. However, for many of the vernacular bridges documented by HAER, including the three discussed in this paper, textbooks contemporary with their construction do not address their unusual structural forms and no original design documents survive. In some cases such as the lattice truss of the Slate Run Bridge, the likely design method is a straightforward extension from a double-intersection Warren truss presented in textbooks. The results of the approximate analysis method can be compared to those of a modern analysis to evaluate the accuracy of the 19th century method. For more unusual structural systems, such as those of the Bluff Dale or English Center Bridges, no approximate methods could be identified, but modern analysis still reveals the true structural behavior of the bridge. The sophistication of the bridge design can be evaluated by accurately calculating member stresses and comparing them to typical 19th century allowable stress levels. Further, the results of modern structural analysis may suggest possible design methods consistent with 19th century bridge engineering knowledge and capabilities.

Evaluation of unusual design features

Many historic bridges include components that are unusual by modern standards. The intentions of the designers are almost never documented, but structural analysis can provide insight into their purpose and evaluate their effectiveness. For example, the primary cable stays of the Bluff Dale Bridge run continuously from anchorage to anchorage, whereas in a modern cable-stayed bridge the stays extend from deck to tower only (Figure 11). Comparative analysis of the Bluff Dale and modern cable systems revealed a unique advantage for the Bluff Dale system as it prevented large axial tension forces in the stiffening truss. The Upper Bridge at Slate Run has several unusual geometric features-longitudinal asymmetry, five component trusses, and diagonals inclined at less than forty-five degrees (Figures 1 and 2). Engineering study revealed that the asymmetry was a result of the bridge having been designed for a different location, a fact not otherwise documented in the historical record.

Study of related structural forms

Full appreciation of the engineering and historical significance of bridges requires consideration of wider traditions of bridge design and construction. Although the influence of prominent, landmark bridges on the design of vernacular bridges is often unclear. Engineering study of the unusual stiffening system of the English Center Bridge led to the consideration of twelve possible stiffening methods. Comparative structural analyses revealed subtle differences in their behavior despite visually similar forms. The unusual cable-stayed system of the Bluff Dale Bridge was compared to other stayed and parabolic cable bridge forms, revealing the merits of its design.

Upper Bridge at Slate Run

The Upper Bridge at Slate Run is a wrought-iron lattice (or multiple intersection Warren) truss built in 1890 by the Berlin Iron Bridge Company over Pine Creek in Lycoming County, Pennsylvania. Major flooding occurred in early June 1889 on many of the rivers in central and western Pennsylvania, destroying bridges and interrupting rail service. The present bridge was constructed in the wake of this flooding to serve the prosperous timber industry of the area and

to provide access to the Pine Creek Railroad, which paralleled the river course and stopped near its east end. The bridge spans 202′–10″ (61.82 m) with a total of 19 panels. The panel at the east end of the bridge is 13′–6″ (4.11 m) long; the panel at the west end, 9′–0″ (2.74 m); and the remaining 17 panels of equal length, 10′–7″ (3.23 m) (HAER PA-460).

The use of a riveted lattice truss for the Upper Bridge is unusual. The lattice form was commonly used for railway bridges on the New York Central Railroad (NYCRR) system from about 1859 to 1890, favored by the engineers Howard Carroll and Charles Hilton (HAER PA-478). The NYCRR was active in northern Pennsylvania during the 1880s and 1890s and eventually acquired the Pine Creek line in 1899. However, the bridge alignment and surrounding topography preclude a rail line passing over the bridge at its present location. In addition, Berlin Iron Bridge rarely built lattice trusses; the only other known example is the Rice Farm Road Bridge in Dummerston, Vermont (1892, 198′–4″, 60.45 m span) used to carry the heavy loads from a nearby stone quarry (Rice Farm Road Bridge 1995).

The lattice truss of the Upper Bridge contains five superimposed Warren trusses (Figure 2), an unusual construction as most lattice trusses were composed of two or four component trusses. A lattice truss in

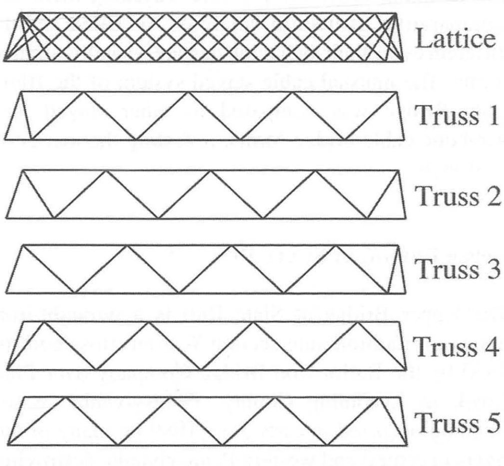

Figure 2
Five component Warren trusses of the Upper Bridge at Slate Run (HAER PA-478)

Fredricksburg, Indiana has recently been identified composed of three component trusses (Barker 2002). The process for determining the number of component trusses required for a given lattice truss remains an outstanding question regarding their design. The engineering analysis of this bridge assessed the behavior of its highly indeterminate lattice system, evaluated the accuracy of an approximate design method, calculated of member stresses, and provided insight into its asymmetric longitudinal geometry (HAER PA-478).

Finite element analysis of the Upper Bridge with a concentrated live load reveals the distribution of axial force in the lattice system (Figure 3). The degree of load sharing depends on the location of the concentrated load—loads applied near the center of the bridge result in more load sharing; those applied near the bridge supports, less. The structural analysis also provides accurate member stresses for various load conditions. With a total dead load of 125 kips (556 kN) and a live load of 1500 lb/ft (22 kN/m), approximating a railroad design loading, the member stresses range from 5 to 10 ksi (35–70 MPa). Allowable stresses for railway bridge design in the late 19th century were about 10 ksi (70 MPa) for members in tension and 6 to 8 ksi (40–55 MPa) for members in compression (Vose 1878). Thus, the modern structural analysis demonstrates that the bridge could have carried railroad traffic. However, it is also possible that the heavy load capacity demanded by the local logging industry and the proximity of railway interests led the designers to adapt the lattice form for use on a roadway bridge, as was done for the Rice Farm Road Bridge (HAER PA-478).

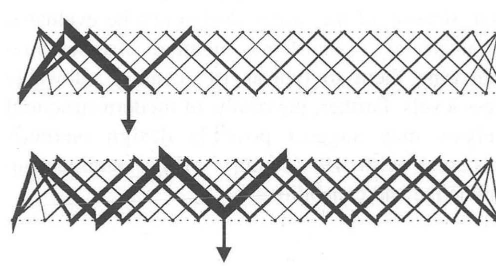

Figure 3
Distribution of member forces in lattice due to unit live load. Line thickness is proportional to axial force.) (HAER PA-478)

As a single structural system the lattice truss of the Upper Bridge is highly indeterminate, but each of the five component trusses is statically determinate. Nineteenth-century bridge engineering textbooks recommended designing multiple-intersection Warren trusses by analyzing each statically determinate component truss independently, considering only the loads which are applied to it. (Merriman and Jacoby, Part I [1888] 1922, Johnson et al., Part I [1893] 1914). The forces in the diagonals are obtained directly from each of the statically determinate analyses, and the forces in the chords and endposts, shared by all component trusses, are the sum of the forces from the individual analyses. Applying this method to the Upper Bridge reveals that this approximate method is reasonably accurate when compared to a modern indeterminate analysis. The approximate method underestimates member forces by less than 10%, a difference well within acceptable limits for 19th-century design. In some members the approximate method overestimates the force by as much as 15%, resulting in some inefficiency, but providing a conservative design.

The longitudinal geometry of the Upper Bridge is asymmetric with end panels of unequal length as described above. In its present configuration there is no clear engineering reason for this asymmetry. However, end panels of different lengths have been used as a convenient means of adapting truss bridges to skew crossings (Figure 4) (Merriman and Jacoby, Part I [1888] 1922). The asymmetric end panels result

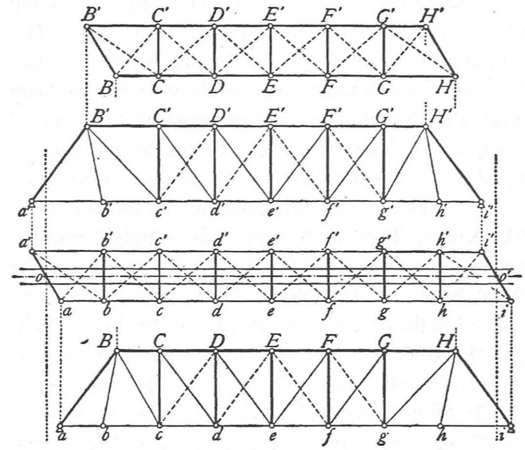

Figure 4
Schematic design of a truss for a skew crossing (Merriman and Jacoby, Part I 1922)

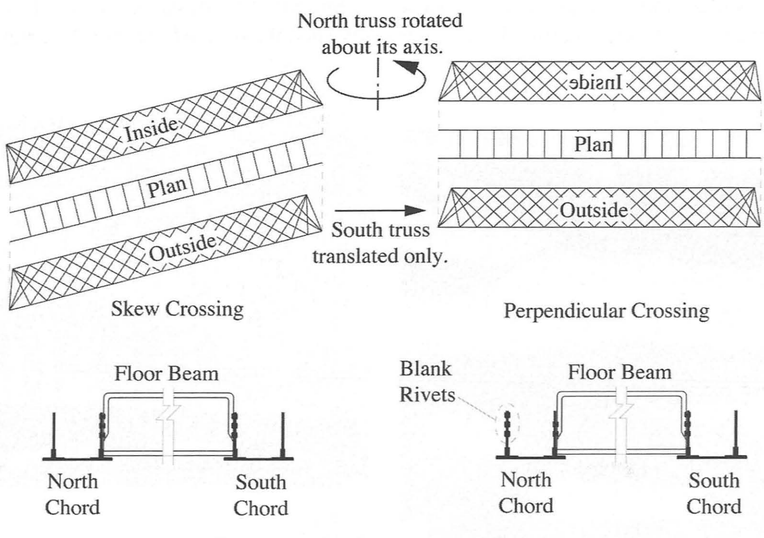

Figure 5
Transformation from skew to perpendicular crossing by rotation of one truss of Upper Bridge at Slate Run

in the alignment of the interior panel points so that the floor beams meet the lower chord at right angles. The asymmetric geometry of the Upper Bridge at Slate Run, as it stands today, is a result of its having been originally designed for a skew crossing of about 14 degrees. To adapt the skew bridge to the present right-angled crossing, the north truss was rotated such that its original inside face appeared on the outside of the bridge. The south truss was simply relocated without any rotation (Figure 5). To reconnect the floor beam to the lower chord, new holes were reamed in the original outside plate of the lower chord and the floor beam riveted to the chord plate. The original inside plate now had six existing holes, which previously held rivets to connect the floor beam, that were no longer necessary. These non-functional holes in the lower chord of the north truss were filled with blank rivets (Figure 6). The lower chord of the south truss has no such holes or blank rivets.

Lower Bridge at English Center

The Lower Bridge at English Center is an example of an uncommon type of suspension bridge with eye-bar chains and full trussing between chain and deck. The surviving Lower Bridge, and its no longer extant twin Upper Bridge, were constructed across Little Pine Creek at English Center, Lycoming County, Pennsylvania, by Dean and Westbrook in 1891. These bridges were among the first built by Dean and Westbrook, as prior to 1891 they served as agents for the Phoenix Bridge Company. Like the Slate Run Bridge, the English Center Bridges were constructed to replace a span destroyed in the floods of June 1889, and they served local industries of logging and tanning. The Lower Bridge has a span of 300′–0″ (91.44 m) divided into 12 equal panels of 25′–0″ (7.62 m).

The hybrid structural system of the Lower Bridge at English Center allows loads to be carried by either truss action or suspension behavior. As a truss, the eye-bars serve as the upper chord, the deck girder as the lower chord, and the diagonals as the web members. As a suspension structure, the eye-bars serve as the primary load carrying mechanism, while the slender deck provides stiffness to the eye-bars. The dominant mode of behavior has been debated by engineers and historians in the past. Some have compared it to double-cantilever bridges with parabolic upper chords, such as the Northampton Street Bridge in Easton, Pennsylvania (Figure 7). Others have suggested that it behaves as a short-span suspension bridge, and the diagonal bracing is superfluous (HAER PA-461).

Structural analysis of the English Center Bridge revealed the true behavior of the bridge and provided

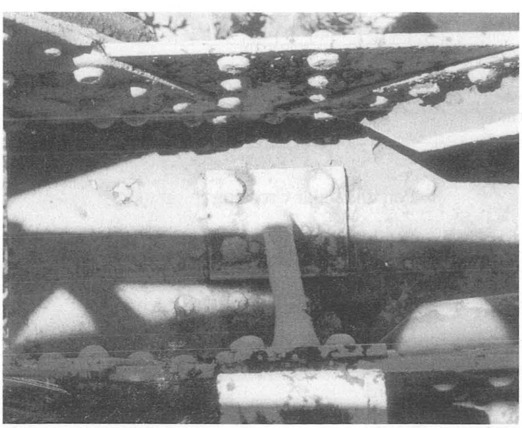

Figure 6
View looking down on lower chord of north truss showing blank rivets on outer plate of Upper Bridge at Slate Run

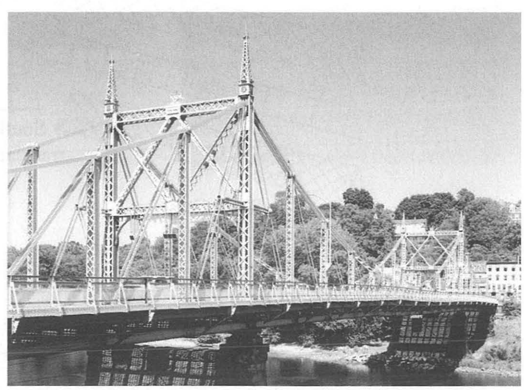

Figure 7
Northampton Street Bridge, Easton, Pennsylvania—a double cantilever bridge constructed with the visual form of a suspension bridge (HAER PA-502, photo 3; Jet Lowe, photographer)

insight into the designer's intentions. A two-dimensional, linear elastic FE model of the Lower Bridge was defined using structural analysis software. Under dead load of approximately 10 kips (44.5 kN) at each panel point, the girder has a significant axial force that increases from the ends toward mid-span, while the axial forces in the upper chord eye-bars increase dramatically from mid-span toward the towers (Figure 8). This force distribution is typical of an inverted two-hinged, trussed arch. This behavior requires the diagonals to have been tensioned during construction prior to the removal of any falsework. Without pretensioning of the diagonals, the dead load would be carried entirely through the eye-bar chain, as for an unstiffened suspension bridge. Unfortunately no details regarding the construction of the bridge survive.

The force distribution due to the live load is somewhat more complex (Figure 8). A unit live load of 1 kip (4.45 kN) was applied to each lower chord panel point in turn. In the panels immediately adjacent to the live load, the load is carried by truss behavior, with the upper chord in compression and the lower chord in tension. Further away from the point of load application, the lower chord is in compression and the upper chord in tension. A qualitatively similar force distribution was observed for all positions of live load application. The live load behavior more closely resembles that of a variable depth truss rather than a suspension bridge, for which the eye-bar chain would remain in tension throughout its length.

The Lower Bridge at English Center was load tested with a highway maintenance truck of total weight 18.6 kips (83 kN) and selected members were instrumented to measure strain or displacement. Load testing captured the true three-dimensional behavior of the bridge, and eliminated assumptions of idealized member behavior and connection properties present in the FE analysis. Live load influence lines

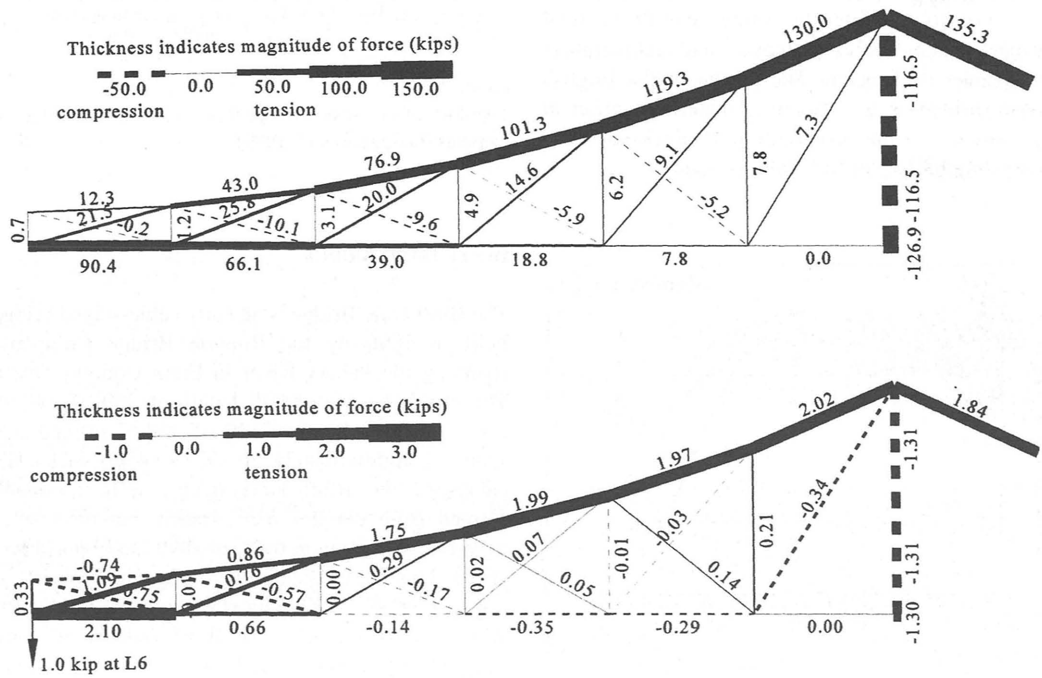

Figure 8
Axial forces in members of English Center Bridge due to dead load (above) and unit live load at mid-span (below). Line thickness is proportional to axial force. (HAER PA-461)

summarize the results of the load testing by showing the manner in which the axial force in a given bridge member changes as the live load is moved across the span. In general the load tests qualitatively confirmed the structural behavior inferred from the FE analyses. Figure 9 shows the influence line for the vertical member closest to the north tower. The load test verified that this vertical member is in compression when the live load is positioned within about 100' (30 m) of it. Other vertical members were also found to carry compressive forces for certain live load locations based on both the load tests and FE models. The vertical members of the Lower Bridge are formed from angles and lacing bars, and those closest to the towers have a box-shaped cross-section—a cross-sectional shape that would only be used for members in compression. The designers correctly expected some compression in these members and most likely designed the bridge as an inverted arch (HAER PA-461, Spivey 2000).

A geometrically non-linear analysis of the Lower Bridge form with no diagonal trussing demonstrated that a substantially greater deck stiffness would be necessary to provide the same overall vertical stiffness as the existing system of diagonal members and slender deck beam. The design of the English Center bridge is an effective, materially efficient alternative to a conventional deck-stiffened suspension bridge for the 300-foot span.

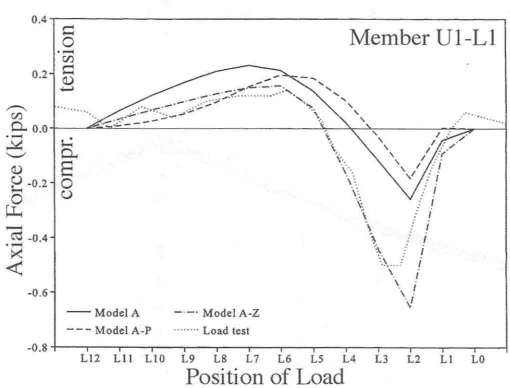

Figure 9
Live load influence lines for several structural models and load test of Lower Bridge at English Center (HAER PA-461)

The study of the Lower Bridge at English Center led to a wider study of the various methods of providing stiffness to the suspension bridge form. Although a deck truss or girder were the most common methods to provide stiffness, many other techniques have been conceived and successfully constructed in the 19th and 20th centuries. (Figure 10) (Gasparini et al. 1999).

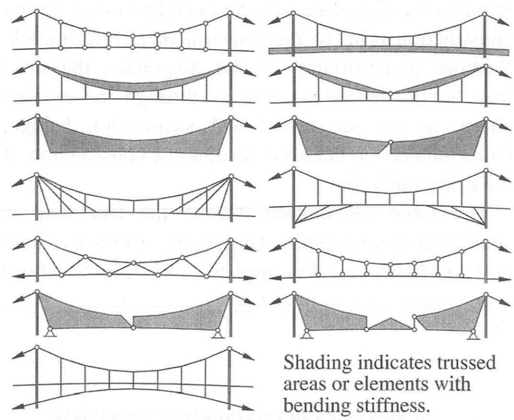

Figure 10
Typology of suspension bridge forms with various stiffening methods (Gasparini et al. 1999)

Bluff Dale Bridge

The Bluff Dale Bridge is an early cable-stayed bridge built in 1890 by the Runyon Bridge Company, spanning the Paluxy River in Erath County, Texas. The bridge has an overall length of 200′–0″ (60.96 m), with a main span of 140′–0″ (42.67 m) and side spans of approximately 30′–0″ (9.14 m) each. The cables of the Bluff Dale Bridge were originally formed from parallel wire strands, but have since been replaced by wire rope. In 1899 the Flinn-Moyer Bridge Company repaired the bridge and replaced its original wooden stiffening truss with the metal truss which survives today. The Bluff Dale is the most complete surviving example of several cable-stayed bridges based on a patent of Edwin E. Runyon. The Barton Creek Bridge has been positively identified as a Runyon bridge, but historical photographs reveal

that others were built as well. HAER has documented a rich tradition of cable-supported bridge construction in Texas, primarily parabolic cable bridges stiffened by trusses or inclined stays (HAER TX-98). Runyon's use of the purely stayed form is unique and historically significant. The engineering and historical reasons for Runyon's use of the cable-stayed form are considered in more detail in Buonopane and Brown (2003).

The Bluff Dale Bridge combines three structural systems—cable stays, horizontal deck cables and a stiffening truss. Further, its cable systems differ from those of a modern stayed bridge (Figure 11). The relative vertical stiffnesses of the three systems determines the distribution of applied gravity loads. Because the stiffness of a cable system is dependent on the magnitude of tension in its cable elements, geometrically non-linear analysis is required. A non-linear mathematical model of a bridge with a single continuous stay, horizontal deck cable, and a three-span stiffening truss was developed in order to identify the most important load carrying mechanisms of the Bluff Dale Bridge. The results of this analysis demonstrated that the horizontal deck cable does not contribute to the gravity load capacity of the bridge. Runyon's patent suggests that the horizontal deck cables were intended to function as longitudinal stringers, directly supporting the transverse bridge decking. Although historical photographs suggest that Runyon's bridges included more typical timber stringers and decking (HAER TX-104).

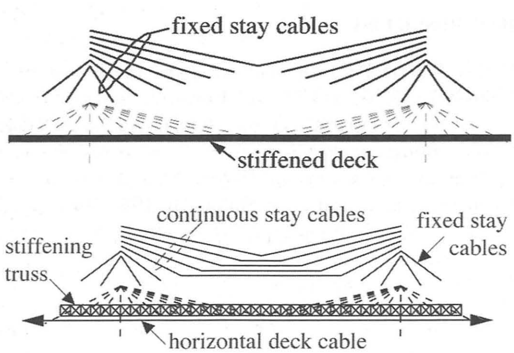

Figure 11
Exploded views of the cable systems of the Bluff Dale Bridge (above) and a modern cable-stayed bridge (below)

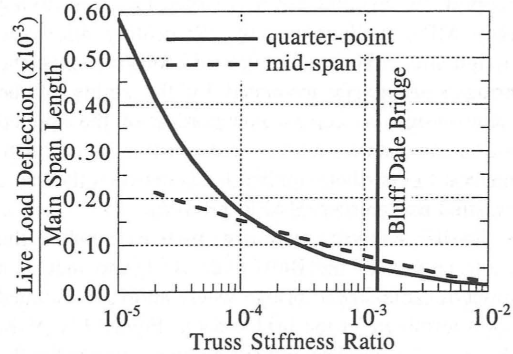

Figure 12
Effect of truss stiffness on live load deflection for Bluff Dale Bridge

Based on the mathematical model several non-dimensional parameters were identified that govern the behavior of the bridge. The most significant parameter is the ratio of truss stiffness to cable stiffness. For small truss stiffness ratios the response of the bridge to live loads is highly non-linear, while for large stiffness ratios the response is essentially linear. Variation of the truss stiffness ratio across a wide range of possible values shows the influence of this parameter on the live load deflections of the bridge calculated by FE analysis (Figure 12). For a live load near the quarter-point and a truss stiffness ratio less than 1×10^{-4}, an increase in truss stiffness results in a significant reduction in deflection. For truss stiffness ratios above 1×10^{-3}, an increase in truss stiffness has a much smaller effect in reducing deflections. Based on this analysis, the truss of the Bluff Dale Bridge (ratio of 1.3×10^{-3}) is stiffer than necessary and a somewhat inefficient use of material, confirming that the bridge was originally designed by some approximate method.

Non-linear FE models of the Bluff Dale Bridge were used to estimate stresses in the bridge members when subjected to dead load and live loads. The maximum bending stress in the truss of the Bluff Dale Bridge is about 6.1 ksi (42 MPa), while the yield strength for wrought iron pipe is about 25 ksi (170 MPa). Allowable design stresses for the late 19th-century would have been about one-third to one-fourth of the yield. In comparison, if the same dead and live loads were applied to an identical truss with no cable

support, the maximum stress would be about 20 ksi (140 MPa) well above any 19th-century allowable design stress. This comparison demonstrates that the bridge's designers accounted for the ability of the cable system to carry some portion of the applied loads, and that they must have used some approximate method to distribute applied loads between the cable system and the truss (HAER TX-104).

The FE analyses were also used to compare the cable pattern of the Bluff Dale Bridge to that of a modern cable-stayed bridge where all of the inclined stays terminate at the bridge deck (Figure 11). Most aspects of behavior were surprisingly similar for the two cable patterns. The only significant difference in behavior was the axial force distribution in the truss due to the uniform dead load. The Bluff Dale Bridge's continuous stay cables result in constant axial tension of about 1130 lb (5 kN) in the center of the bridge, whereas the modern cable pattern results in a maximum tension of 9940 lb (44 kN). The lower axial tension in the truss of the Bluff Dale Bridge results in significantly lower stresses in the truss chords. In a modern cable-stayed bridge, the axial force in the deck is controlled through construction methods and cable tensioning, typically resulting in compression throughout the deck. Such modern construction techniques were not available to the builders of the Bluff Dale Bridge.

CONCLUSIONS

This paper has reviewed methods of engineering analysis applied to the documentation of historic bridges. In these three examples documented by HAER, the results of the engineering analysis provided significant insights into the history and design of the bridges. Modern engineering analysis allows accurate analysis of indeterminate and non-linear structures, unavailable to 19th century bridge designers. Approximate design methods and unusual design features can be evaluated with modern engineering analysis. Comparison of member stresses with allowable values from the 19th century, provides the engineering historian insight into the sophistication of the design. Just as proper historical documentation must consider the wider cultural context of a bridge, engineering study should consider the bridge as part of more prominent trends in structural design. Two critical engineering issues —construction sequence and member prestressing— cannot be determined by modern analysis and remain outstanding issues in the study of historic bridges.

Engineering analysis is a critical component of the study of historic bridges and structures. The surviving structure is a historic document that can be «read» through the use of modern structural analysis, providing insights into the designers' intentions and the efficiency of their designs. This both enriches historical understanding and has the potential to contribute to the innovative design of modern structures. The Historic American Engineering Record continues to include engineering analysis as a part of its bridge documentation projects. Recently HAER has completed historical and engineering study of four structural forms of American wooden covered bridges. Documentation and research of covered bridges is expected to continue over the next two years.

ACKNOWLEDGEMENTS

The research summarized in this paper was supported by the Historic American Engineering Record, and the Pennsylvania and Texas Departments of Transportation as part of summer documentation programs. The authors gratefully acknowledge the historical research of Mark Brown and Blythe Semmer, and load testing performed by Thomas Boothby.

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History and engineering analysis of the 1890 cable-stayed Bluff Dale bridge

Stephen G. Buonopane
Mark M. Brown

During the 1890s, Edwin E. Runyon and William Flinn constructed a group of innovative cable-stayed bridges in north central Texas. The Bluff Dale Bridge (Figures 1 and 2), the most complete example of the collaboration of these designers, was originally constructed in 1890 based on a bridge system patented by Runyon. Although renovated and relocated, Bluff Dale is the second oldest surviving cable-stayed bridge in Texas and possibly in the United States as well. Historians have recently identified a slightly earlier surviving Runyon cable-stayed bridge —the Barton Creek Bridge of 1890, completed several months prior to Bluff Dale (Figure 3). The towers, cables and floor beams at Barton Creek survive in their original form, thus providing important information on original construction details that no longer survive at Bluff Dale.

The Bluff Dale Bridge is a significant example of a local cable-supported bridge building tradition of the late 19th-century and demonstrates the inventiveness and proficiency of the designers. Runyon and Flinn responded to the engineering challenges of bridge design and construction with inventive solutions different from the designs of more prominent suspension bridges. This paper explores the significance of Runyon's bridges in the context of the development of 19th century cable-supported bridges, including historical precedents and engineering analysis of the unusual cable-stayed design.

In 1890 the commissioners of Erath County, Texas accepted a $4,200 bid by the Runyon Bridge

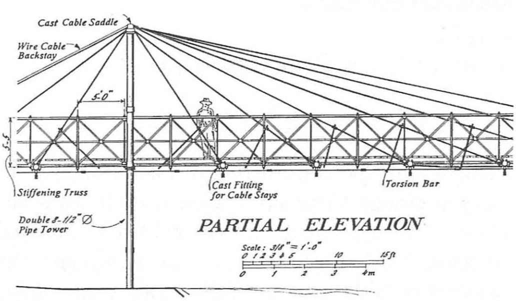

Figure 1
Bluff Dale Bridge with metal truss of 1899 (HAER TX–36, sheet 3; Erick McEvoy, delineator)

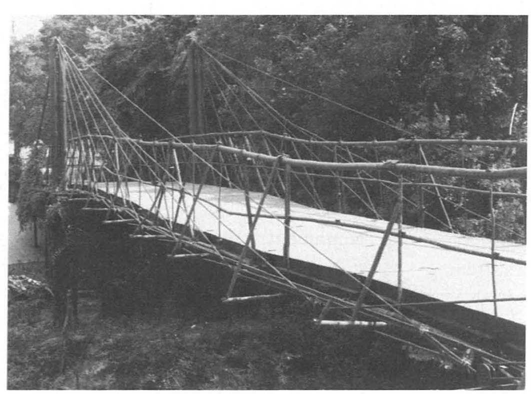

Figure 2
Bluff Dale Bridge in 1996 (HAER TX–36, photo 3; Joseph Elliott, photographer)

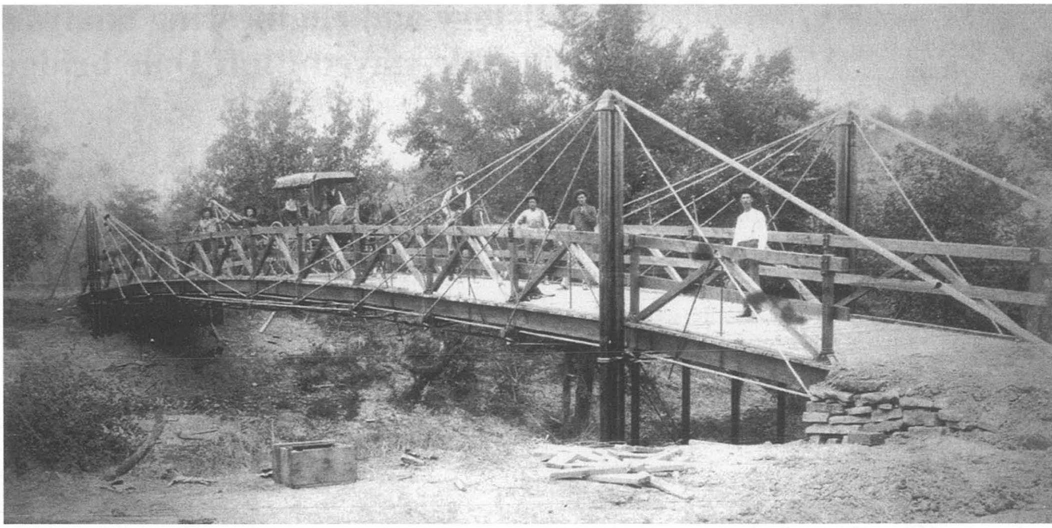

Figure 3
Barton Creek Bridge (HAER TX–36, photo 12)

Company for the construction of three bridges. The Runyon Bridge Company consisted of Runyon and Flinn, although there is no evidence of later collaborations between the two. Flinn built many other suspension bridges in Texas, and the Flinn-Moyer Company completed repairs to the Bluff Dale Bridge in 1899, replacing the original wooden truss with the surviving metal truss of pipe and rod sections. The

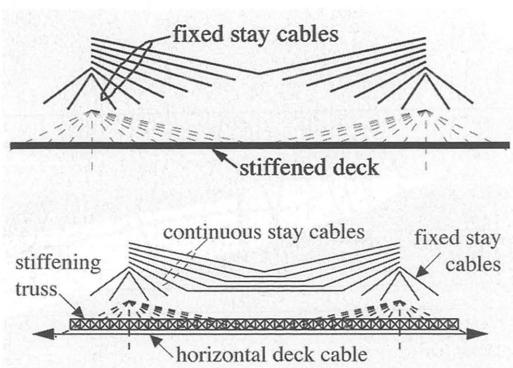

Figure 4
Exploded views of the cable systems of the Bluff Dale Bridge (above) and a modern cable stayed bridge (below)

Bluff Dale Bridge has a main span of 140′ (42.67 m) and side spans of approximately 30′ (9.14 m) each. The spans are supported by cable stays of two types—fixed and continuous, arranged in a fan pattern (Figure 4). In contrast, all stays of a typical modern cable-stayed bridge are fixed to the deck. The stay cables were composed of heavy gauge parallel wire strands. The stays of the Barton Creek Bridge contain about 30 strands of No. 9 gauge wire (0.148″, 3.75 mm diameter), and the builders of Bluff Dale probably used No. 9 wire, as it was the most common size for bridge construction (HAER NJ–132). The wires of the five continuous stays are bundled together to form the backstay. All of the stays of the Bluff Dale Bridge have been replaced by modern wire rope, probably at the time of its relocation in 1935. Additional description and drawings of the Bluff Dale Bridge can be found in HAER TX–36 and Brown (1998).

Runyon's patents and the Bluff Dale Bridge

The Bluff Dale Bridge follows the concept of Runyon's 1888 patent (No. 394,940) for a cable-stayed bridge. The patent drawing (Figure 5) shows three panel points, but if extrapolated to include additional

panel points, the resulting cable pattern could become either that of Bluff Dale or the «crossing fan» pattern used elsewhere by Runyon (Figure 6). The 1888 patent also includes horizontal «deck cables» that run longitudinally beneath the bridge deck. The center deck cable rests in saddles attached to the floor beams with no positive connection. The two outer cables sit in castings at the end of each floor beam, but U-bolts secure the cables to the floor beams. The patent description implies that the deck cables were the first elements to span the river during construction, providing an attachment point for the needle beams. The patent also states that the deck cables could replace longitudinal stringers, yet historical photographs show the more traditional wooden stringer and decking system on some Runyon bridges (Figures 3 and 6).

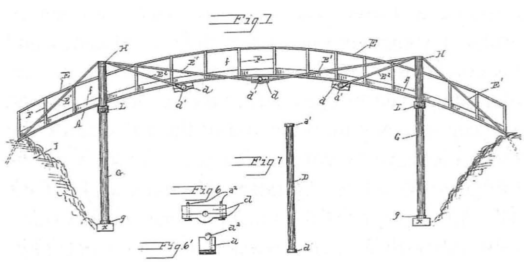

Figure 5
Elevation of cable stayed bridge from Runyon's patent (No. 394,940)

Figure 6
Unidentified Runyon bridge with «crossing fan» cable pattern (HAER TX–36, photo 14)

The transverse floor beams, or «needle beams,» are based on an 1889 patent (No. 400,874). They are composed of a horizontal pipe section and a lower chord of about 25 strands of No. 9 wire, separated by three vertical castings. The bowstring action of the beams provides substantial rigidity and bending resistance. The ends of the needle beams are fitted with a complex set of castings, which include attachment points for the bowstring cable, the deck cables, lateral X-bracing and the main stay cables (Figure 7). The needle beams of Bluff Dale survive in original condition including the parallel wire lower chords.

All of the cable elements of the Bluff Dale and Barton Creek Bridge were tensioned using a twisting device patented by Runyon in 1889 (No. 404,394) (Figure 8). The wires of the cable were separated into two bundles by a small casting (Figure 9). A circular device, clamped to the casting and rotated about the axis of the cable, twisted the two bundles and tensioned the cable. The builders then inserted a metal «torsion rod» into a hole in the casting and braced it against the bridge to prevent unwinding. The twisting device could then be removed and reused elsewhere on the bridge. Pretensioning the cables of a stayed system to remove slackness can provide substantial vertical stiffness to the bridge. This construction technique is instrumental to the success of a stayed bridge and Runyon's use of it

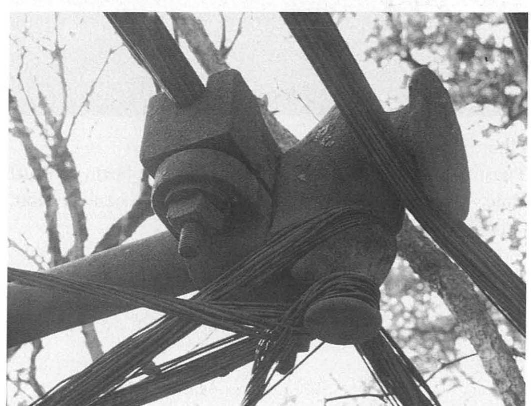

Figure 7
Castings and cables at end of needle beam from Barton Creek Bridge (HAER TX–87, photo 7; Bruce Harms, photographer)

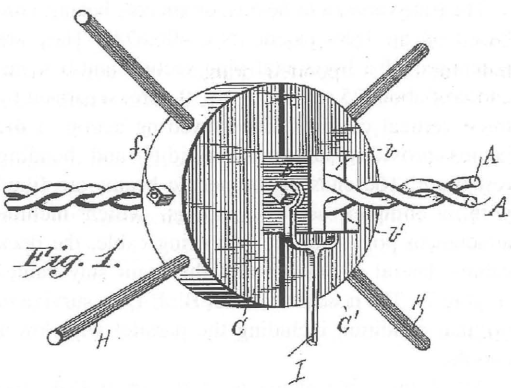

Figure 8
Patent drawing of Runyon's cable twisting device (No. 404,934)

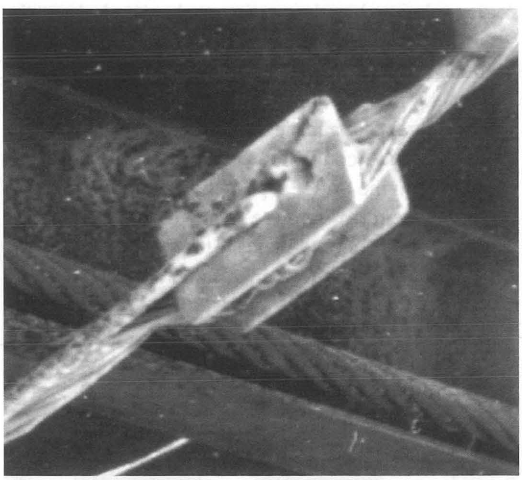

Figure 9
Twisting block on bowstring cable of needle beam of Bluff Dale Bridge (HAER TX–36, photo 6; Joseph Elliott, photographer)

demonstrates his sophistication as an engineering designer.

Development of cable-supported bridge forms

Both trained engineers and empirical builders have shaped the history and development of cable-supported bridges, as they independently translated the principles underlying these structures into a wide variety of bridge forms, from ancient examples in Asia and South America to the more well-documented European examples. In the early 19th century, a wide variety of cable-supported bridge forms were constructed, including stayed, parabolic and hybrid forms, and these bridges used various materials for the stays, including wire cables, wrought-iron chains and solid rods. A group of twelve unusual stayed bridges built in Scotland and England by local builders between 1816 and 1834 demonstrates such variety (Ruddock 1999). In 1823 Navier's *Rapport . . . et memoire sur les ponts suspendus*, the first theoretical analysis of cable-supported bridges, concluded that parabolic suspension bridges were preferable to cable-stayed since their flexibility allowed them to change shape in response to loads. Navier's work widely influenced bridge designers in Europe and the United States, and his conclusions may have contributed to the decline of cable-stayed bridges during the second half of the 19th century. Not until the end of the 19th century did the cable-stayed form re-emerge in France with the bridges of Gisclard, Arnodin and Leinekugel le Coq. The development of the modern cable-stayed bridge, characterized by high strength steel wires and large pretension forces, is generally attributed to Dischinger's work in the 1930s and the construction of the Strömsund Bridge in Sweden in 1955 (Troitsky 1988, Walther et al. 1999).

James Finley is supposed to have built the first chain suspension bridges in the United States about 1800, and Kranakis (1997) details his career and empirical design methods. Josiah White and Erskine Hazard introduced the use of wire cables in 1816 for a pedestrian bridge over the Schuylkill River in Philadelphia (Peterson 1986). The development of the suspension bridge in the United States was played out through the careers of Charles Ellet, Jr. and John Roebling. Ellet favored European design methods with shallow cables and a very lightly stiffened deck, exemplified in his Wheeling Bridge of 1849 (Kemp 1999). Roebling's career and that of his son Washington are well documented elsewhere. Of interest here is the manner in which Roebling's last two bridges —Cincinnati (1866) and Brooklyn (1883)— influenced other bridge builders and captured the imagination of the general public. The

«Roebling system» of parabolic cables, stiffening truss, and inclined stays became an engineering and visual trademark adopted by bridge builders nationwide and favored by public agencies awarding new bridge contracts. In the Ohio Valley, not far from the Cincinnati Bridge, John Shipman built several suspension bridges with inclined stays with spans from 300′ to 560′ (90 m–170 m) between 1852 and 1876. The Roebling Company wrote the specifications for some of these bridges and often supplied the wire that was used (Simmons 1999). An 1877 advertisement for Shipman's New York Bridge Co. includes an image of «the celebrated "Roebling" Steel Wire Suspension Bridge» (Darnell 1984).

Cable-supported bridge tradition in Texas

The cable-supported bridge tradition in Texas begins with Thomas Griffith's construction of the Waco Bridge in 1870 over the Brazos River (HAER TX–13, TX–98). The relationship of Griffith to the Roebling Company is unclear, but he may have worked on the construction of Roebling's Niagara Bridge in the 1850s. He later independently built two suspension bridges in Minneapolis in 1855 and 1875. The Waco Bridge clearly bears the mark of Roebling influence —parabolic cables, inclined stays and a deep stiffening truss— and we know that the builders consulted with and purchased materials from the Roebling Company. Ease of transport of materials and on-site construction are among the technical reasons that may have favored the selection of a suspension bridge at Waco, instead of a fabricated metal truss (Brown 1998). Indeed, transport of materials and fabrication still concerned Griffith in 1883 when he patented a suspension bridge system (No. 285,257) «composed entirely of pieces of moderate length and weight which can easily be carried by men or pack-mules, and which when once delivered at the site of the proposed structure can be easily and cheaply put together». The Waco Bridge shaped Texans' concept of «bridge» —two different Texas bridge companies used its image in their advertising (HAER TX–36).

The prominence of the Waco Bridge, and the popularity of suspension bridges in general, contributed to the emergence of this bridge form in north central Texas. Economic considerations, such as cost of materials, ease of transport and construction, also were significant factors in making bids for cable-supported bridges competitive with, and often cheaper than, truss alternatives. Further, the ability to span rivers without a mid-stream pier provided an additional technical advantage for cable-supported bridges; Texas river-courses are notorious for flash floods and poor soil conditions unfavorable for foundations. This is the context in which Runyon proposed the cable-stayed form, intentionally departing from the parabolic form, even while such bridges were successfully being built at nearby sites in Texas.

Joseph Mitchell

Although there are few obvious precedents for Runyon's bridges, the work of Joseph Mitchell bears striking similarities to that of Runyon. Little is known of Mitchell's work in Texas, other than records from four different Texas counties indicating bridge contracts between 1886 and 1888, including one for repair of bridges he had previously built (HAER TX–98). In 1887 Mitchell received a patent (No. 368, 483) for a bridge with a primary structural system best described as a wooden truss, but featuring deck cables similar to those later used by Runyon. Mitchell used five galvanized wire cables to replace the longitudinal stringer beams, allowing the transverse floor boards to rest directly on the cables. The deck cables were tensioned by twisting with rods which were then braced against an adjacent cable to prevent unwinding. Mitchell describes his patent as providing «a bridge which is cheap in construction . . . and also to do away with the floor-beams and substitute therefor cables which are so constructed that they may be tightened without the necessity of removing the floor-boards . . . » In 1890 Mitchell received a patent (No. 440,490) for bridge construction that includes a bowstring beam with a lower chord of twisted wires. Mitchell also describes a «lozenge shaped block» used to twist the wires, very similar to that later used by Runyon.

The last known reference to Mitchell's work in Texas occurs on 10 September 1888 in Cooke County, the same day on which Runyon was awarded a commission. Shortly thereafter on 27 October 1888, Mitchell was awarded a bridge contract in Fulton

Figure 10
Whitewater River bridge by J. Mitchell at Richmond, Indiana (Bridges over . . . 1899)

County, Indiana, for «Three Cable Bridges of his Patent of August 16, 1887.» One of these bridges, completed in 1889, spanned the Whitewater River in Richmond, Indiana (Figure 10). An advertisement for Mitchell's bridges, illustrated with a photograph of the Whitewater River bridge, stated: «It cost $2,150 for sub and superstructure complete, or about one-fourth of the cost of other iron bridges, and equal to them in strength and superior durability, as there is no wood except the floor, and it rests on galvanized steel cables, so anchored that it cannot be washed away.» The bridge at Richmond had a main span of 150' (45.7 m) with six equal panels, pipe tower bents, a stiffening truss fabricated from strap-iron and round sections, and presumably the Mitchell-patented deck cables. Based on the only known photograph of the Whitewater Bridge, its stiffening truss was discontinuous at the towers and the main span exhibited a noticeable sag. Reportedly a person exciting the bridge at its quarter-point could produce vertical undulations of 12» to 18» (30–45 cm) (Bridges over . . . 1899). These observations suggest that the stay cables were not effectively tensioned and the deck not sufficiently stiffened by the truss. Perhaps similar behavior on Mitchell's earlier bridges in Montague County, Texas led county commissioners in 1888 to order him «to repair all Bridges built by him in this County» (HAER TX–98). The Whitewater River bridge was destroyed in a flood in 1897.

Edwin E. Runyon

The similarities between the Mitchell and Runyon bridges and their presence in north central Texas circa 1888 certainly suggests that their work influenced one another. Unfortunately the historical record does not indicate whether they were collaborators or competitors. Very little is known about the life and career of Runyon. The primary sources are county records, patents, photographs and a business card. In 1879 Runyon lived in Cooke County and worked as a schoolteacher and shopkeeper. It is unlikely that he received any formal advanced engineering training. Based on the patents issued to him between 1888 and 1893, Runyon lived in several towns in north central Texas and appears to have been a somewhat itinerant inventor. Runyon received six patents related to bridge construction, as well as patents for a cotton cultivator and a lawn mower (HAER TX–36).

ENGINEERING ANALYSIS

Modern engineering analysis of the Bluff Dale Bridge can address several issues of historical importance, regarding the overall behavior of the bridge, its unique stayed form and unusual design features. In the 19th century, the Bluff Dale Bridge would have been designed using approximate analyses and empirical rules. But modern structural analysis can accurately determine forces and stresses in the statically indeterminate bridge form. Geometrically non-linear effects are also included to properly account for the large deformations associated with cable structures. An analytical model was developed that captured the fundamental behavior of the stayed bridge system, and a set of important non-dimensional parameters were identified from the model. The detailed behavior of the bridge was also examined with finite element (FE) models based on the surviving 1899 metal truss. Since the construction sequence and cable pretensioning of the bridge are not known, dead and live loads were applied simultaneously to the complete FE model, providing an upper-bound estimate of forces and stresses in the bridge truss. The complete engineering analysis of the Bluff Dale Bridge is contained in HAER TX–104.

Cable systems

The structural system of the Bluff Dale Bridge can be best described as cable-stayed, although it possesses

two unique features that differentiate it from modern cable-stayed forms —horizontal deck cables and continuous inclined stays (Figure 4). The simplified analytical model with a single continuous stay, deck cable and truss showed that the stay cable carries 42% of the applied vertical load; the truss, 58%; and the horizontal deck cable virtually none of the load (Figure 11) for both symmetric and asymmetric load conditions. In combination with the stiff truss, the deck cables do not contribute to the gravity load capacity of the bridge. Non-dimensional analysis confirmed that the deck cables will not carry gravity load, even with large levels of pretension force.

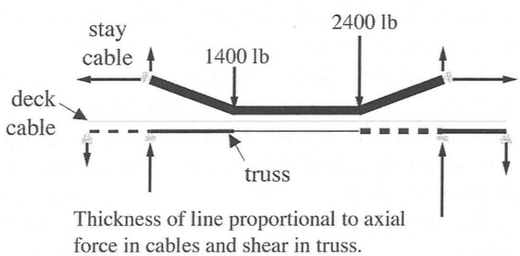

Figure 11
Force distribution in simple model of a stayed bridge composed of a continuous stay, deck cable and truss.

The structural behavior of the continuous stay system of the Bluff Dale Bridge was compared to that of two other possible cable patterns —the crossing fan pattern (Figure 6) and the modern fan pattern (Figure 4). The Bluff Dale stay system uses approximately the same total weight of wire as the crossing fan pattern, but nearly 20% more than the modern fan pattern. The bending moments and deflections of the truss due to a uniform dead load of 140 lb/ft (2.0 kN/m) were found to be remarkably similar. The only significant behavioral difference between the cable patterns appears in the axial force distribution in the truss (Figure 12). The continuous stay cables of the Bluff Dale Bridge result in constant axial tension of about 1130 lb (5.0 kN) in the center of the bridge, while the modern cable pattern results in a maximum tension of 9940 lb (44.2 kN). In a modern cable-stayed bridge, the axial force in the deck is controlled through construction methods and cable tensioning, typically

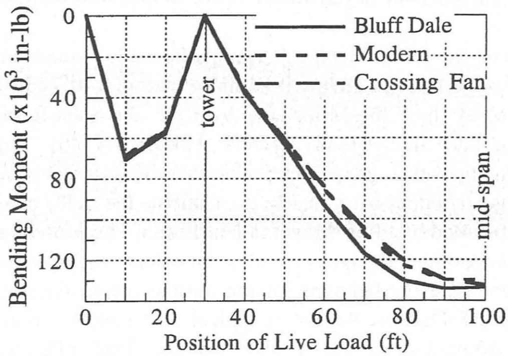

Figure 12
Axial force in truss due to dead load for three cable stay systems

resulting in compression throughout the deck. Such modern construction techniques were not available to the builders of the Bluff Dale Bridge. Large axial tensions in the original wooden truss or the surviving pipe truss could have contributed to loosening of the connections and would have been considered undesirable. The designers of the Bluff Dale cable system may have been aware of this reduction in tension through experience. The live load influence lines of both truss bending moment and vertical deflection were also remarkably similar for the three cable patterns, with no clear behavioral advantage for any of the three systems (Figure 13).

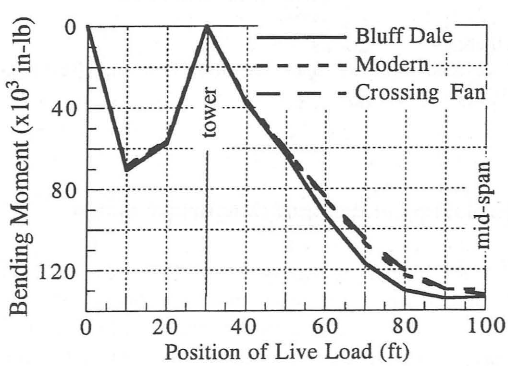

Figure 13
Live load influence lines of truss moment for three cable stay systems

Comparison to parabolic cable suspension bridge

The Beveridge Bridge is a parabolic cable suspension bridge built in 1896 with a main span of 140′ (42.67 m) by the Flinn-Moyer Co. over the San Saba River in San Saba County, Texas (HAER TX–46). The main span length is identical to that of Bluff Dale and its stiffening truss nearly the same as the 1899 truss installed by Flinn-Moyer at Bluff Dale. Therefore, the Beveridge Bridge provides an ideal example to compare the behavior of the unique stayed form of Bluff Dale to that of a typical 19th century truss-stiffened suspension bridge. The live load influence lines show the Bluff Dale Bridge to have slightly smaller vertical deflections (Figure 14). However, the cable system of the Beveridge Bridge is estimated to use about 15% less material than that of the Bluff Dale Bridge, and thus could be considered a more efficient design.

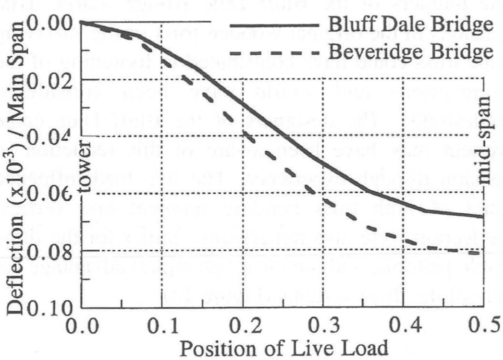

Figure 14
Live load influence lines for Bluff Dale and Beveridge Bridges

Evaluation of the Bluff Dale Bridge design

The FE analyses provide forces and stresses in all bridge members for dead and live load conditions. Based on the dead load truss moments, plus moments due to a live load at midspan, the maximum bending stress in the truss is about 6100 psi (42 MPa). The yield point of wrought iron typically ranges from 25 to 35 ksi (170–240 MPa) and typical design practice at the turn of the century would have allowed a working stress of about one-fourth of the elastic limit (Withey and Aston 1926). If the truss of the Bluff Dale Bridge were to be used with no cable support, the bending stresses would be as large as 20 ksi (140 MPa), certainly well above a typical design level for the late 19th century. Although the designers of the Bluff Dale Bridge were not able to perform the detailed calculations necessary for a complete analysis of the cable-supported truss, they may have been capable of calculating the stresses for the unsuspended three-span truss. The Bluff Dale Bridge was designed, and its truss members proportioned, with the intent that a significant portion of the load would be carried by the cable stay system. Additional non-dimensional analysis of the relative stiffness of the truss and cable stay system indicated that the truss of the Bluff Dale Bridge is significantly stiffer than necessary to maintain vertical deflections within serviceable limits, further confirming that the bridge was originally designed by an approximate or empirical method.

The designers' method of distributing load between the truss and cables is not known. One possible approximate design method for deck-stiffened, cable-supported structures is to design the cable system to carry all of the dead load and to design the truss as an unsuspended span for the live loads only. This method satisfies equilibrium, since all gravity loads are accounted for, and is attractive due to its simplicity. Typically the method results in conservative estimates of truss stresses, and somewhat unconservative estimates of cable stresses, although the strength of drawn wire cables typically was sufficient to accommodate the true stress levels. Conceptually, this method is similar to that used by John Roebling to design his parabolic cable suspension aqueducts:

> The original idea upon which this plan has been perfected, was to form a *wooden trunk*, strong enough to support its own weight, and stiff enough for an aqueduct or bridge, and to combine this structure with wire cables of a sufficient strength to bear safely the great weight of water. (The wire suspension aqueduct . . . 1845)

The construction sequence of the aqueducts would have resulted in an actual load distribution different from that assumed in the design.

Conclusions

The Bluff Dale Bridge is a rare example of 19th century cable-stayed bridge design and a striking part of a larger tradition of cable-supported bridge construction in Texas. Cable-supported bridges provide advantages for construction in remote areas, including ease of ground transport of materials, ease of construction and economic use of materials. The prominence of the long-span suspension bridges of Ellet and Roebling contributed to the adoption of the suspension bridge form for many moderate spans. Runyon's work shows many similarities to that of Joseph Mitchell, but their relationship remains unclear. While all of these influences shaped Runyon's work, his remarkable 1888 patent illustrates a purely cable-stayed form, rather than the more common parabolic form. This and other patents show Runyon's basic understanding of engineering principles, more likely gained through experience than formal education. Runyon and bridge-builder Flinn were able to design and construct several successful bridges on this innovative scheme. Especially crucial to the performance of Runyon's bridges was the twisting of cable elements to remove slack and perhaps provide some pretension.

In light of the modern cable-stayed form, two design features of Bluff Dale are especially intriguing —the horizontal deck cables and continuous stays. The deck cables, used in parallel with a stiff truss and stays, do not carry vertical loads, but they could have supported the transverse flooring and were probably useful during construction. The continuous stay cables prevent the transfer of axial tension to the stiffening truss, which could have been detrimental to the integrity of the stiffening truss. The designers of the Bluff Dale Bridge clearly accounted for some distribution of applied load between the cable and truss systems, although the nature of their approximate design method remains unknown.

Combined historical and engineering study of vernacular bridge design can reveal innovative solutions, and suggests that the history of cable-supported bridges is much deeper and richer than the canon of famous and monumental examples. Historical and engineering study may also inform and improve the design of modern structures. A recent innovative design for the Maumee River Bridge in Toldeo, Ohio, by Figg Bridge Engineering employs cable stays which are continuous through the towers with their ends anchored at the bridge deck. By not requiring cable anchorages in the towers, this method allows for lighter and more aesthetically pleasing tower designs (Cradle system . . . 2002). This cable system bears great resemblance to the fixed stays used by Runyon on the Bluff Dale and Barton Creek Bridges more than a century ago for the very same reasons.

Acknowledgments

The research summarized in this paper was supported by the Historic American Engineering Record and the Texas Department of Transportation, Environmental Affairs Division as part of a documentation program during the summers of 1996 and 2000. The authors gratefully acknowledge the reviews of this research by Charles Walker of Texas DOT, Dario Gasparini of Case Western Reserve University, Justin Spivey of Robert Silman, P.C. and Sybil Eakin.

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The experimental earthen cottages at Amesbury, England: A long term condition assessment

Richard Burt

Between 1919 and 1920 the British Government, under the direction of the Department of Scientific and Industrial Research, built a series of experimental cottages in Amesbury, England. The experiment was in response to the shortage of available construction materials after the end of World War I. The Cottages were built on land purchased by the Board of Agriculture as part of the Land Settlement (Facilities) Act 1919. This act aimed to provide employment and smallholdings for ex-serviceman returning from the war (Parker 2000, 2). Amesbury was a small community located close to Stonehenge in the west of the country. The experiment consisted of the construction of five experimental cottages built on Ministry of Agriculture and Fisheries land. Three of the cottages were built «to test various old methods of construction which had fallen into disuse, and which it might prove desirable to revive». The remaining two cottages were built «to test certain new methods of constructing floors, roofs, and the like» (Jaggard 1921, 1). It is the former of these cottages that is the focus of this paper.

The three cottages built to test «old methods» of construction used variations of traditional earth construction techniques and they are referred to in this paper by their original numbers from the 1921 report (Jaggard 1921, 6). They were:

- Cottage nº 4, Ratfyn. Built of chalk and straw, and with a Roman tiled roof.
- Cottage nº 5, Ratfyn. Built of chalk-pisé (chalk and soil) and with a slated roof.
- Cottage nº 10. Built of chalk and cement and with a pan-tiled roof.

Two of these experimental cottages were considered of such special architectural interest that they were listed by the Secretary of State for Culture, Media and Sport under the Planning (Listed Buildings and Conservation Areas) Act 1990, on advice from English Heritage. Listing a building afforded it some protection; listing ensured that the architectural and historic interest of the building was carefully considered before any alterations; either outside or inside were allowed.

The focus of this paper is primarily on the earthen walls that form the main load-bearing structure of the cottages. The paper sets out how the walls were constructed and evaluates the condition and development of the three cottages at three subsequent points in time: 1927, 1945 and 2002.

The construction of the experimental cottages

The three experimental earthen cottages were all based on one of the model plans developed by the Ministry. The model plan chosen was known as C,[1] and all three cottages were a variation of this plan (Jaggard 1921, 5). The Ground and Upper floor plans

for cottage n° 4, Ratfyn were reproduced from the plans in the original report using AutoCAD and are shown in Figure 1. Chalk was used as the main walling material for many reasons; it was available on the site and it had previously been used as a building material locally, evident from many surviving cottages in the village (Jaggard 1921, 17). The three cottages used variations of Pisé that utilized a standard form of timber shuttering. The main walls for the three cottages were constructed as follows and are shown in Figure 2.

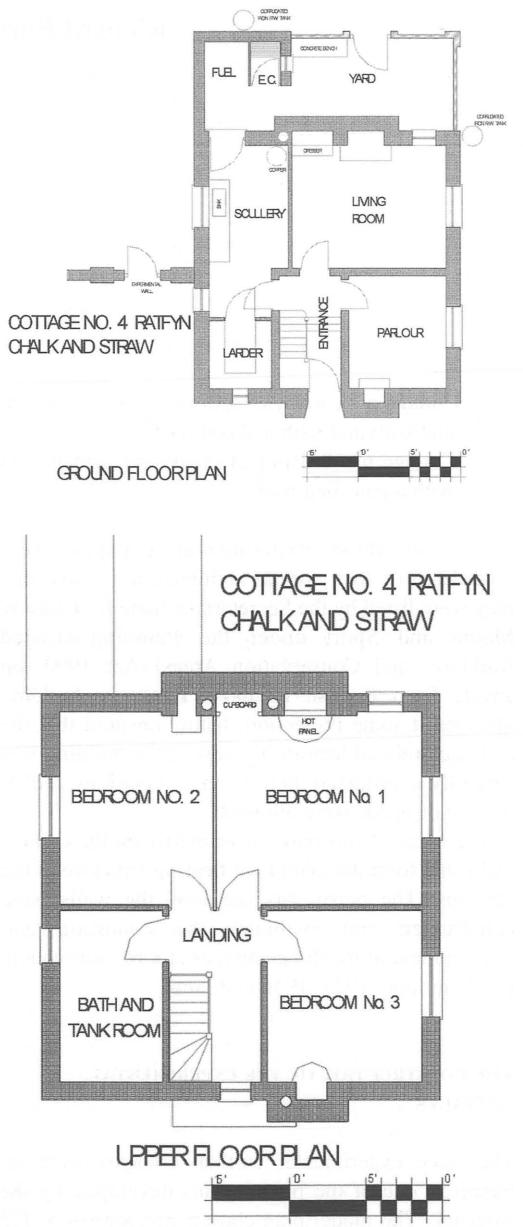

Figure 1
Ground and upper floor plans of n° 4 Ratfyn

Cottage n°4 Ratfyn

The main walls of this cottage were built of a mixture of chalk, straw and water. The larger pieces of chalk were broken down to about 2 in. in diameter. Chalk and straw were then mixed together with enough water to make the mixture plastic. The moisture content of the water was 8% by weight. This mixture was then placed in the wooden shuttering in 3 in. layers and rammed using wedge and heart-shaped rammers. Subsequent layers were added and consolidated to a depth of approximately 18 in.. The shuttering was then removed and the walls were allowed to dry. This process was repeated until the full height of the wall was reached (Jaggard 1921, 20).

The chalk walls were built on a concrete foundation that was 1ft. 9in. deep and 1ft. 6in. wide. A 1 ft. high wall consisting of a half brick internal face and unsnapped flint facing was formed upon the concrete foundation and a double course of slates was used as a damp-proof course. The damp-proof course was positioned 9 in. above the finished ground level (Jaggard 1921, 16). The walls were built 1ft. 5in. thick for the ground floor and this was reduced to 1 ft. 2in. for the upper floor. Rectangular pre-cast reinforced concrete lintols were used over the openings and the window cills. They were formed using an oversailing brick course topped with a double course of plain tiles. The internal surfaces of the wall were finished with a two-coat-lime-sand plaster and the external surfaces finished with a thin coat of lime wash (Jaggard 1921, 9).

Construction of the main walls began at the end of August 1919 and was finished by late January 1920. Most of the work was therefore undertaken in the autumn and winter. The report notes that in ordinary circumstances this type of work should be done in the summer months (Jaggard 1921, 22).

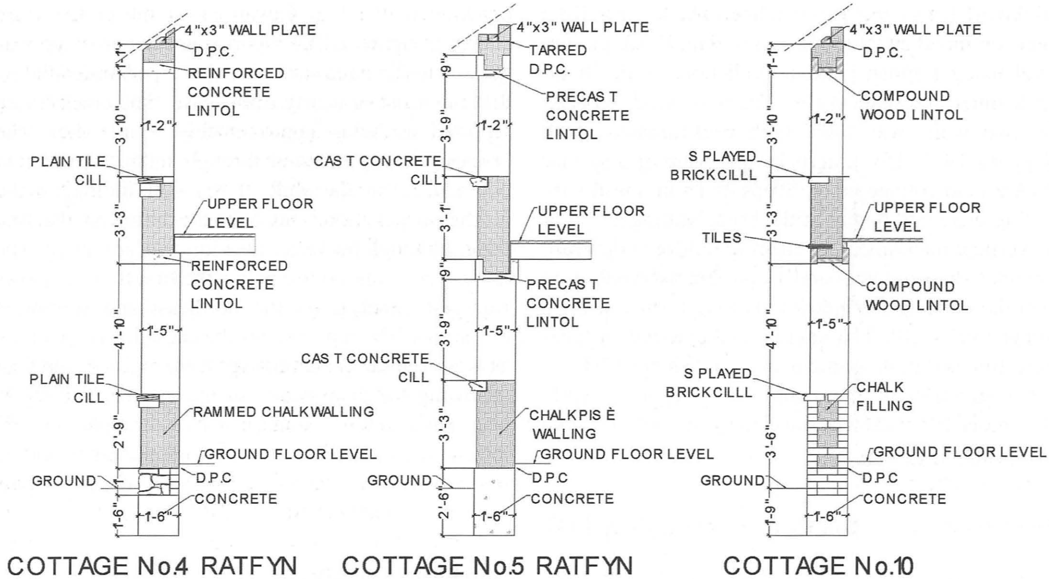

Figure 2
Construction of External Walls of Experimental Cottages

Cottage n° 5 Ratfyn

The main walls of the cottage were built using chalk pisé. Chalk pisé is a mixture of approximately 30% fine soil located underneath the top soil and 70% chalk. No straw or water was added. The mixture was thoroughly mixed together by hand, placed in the shutters, and rammed in a similar manner to the previous cottage. However, this time, the height of the chalk pisé layer was increased by 9 in. to 27 in. (Jaggard 1921, 26).

The chalk walls were built on a concrete foundation that was 2ft. 6in. deep by 1ft. 6in. wide and extended approximately 9 in. above finished ground level. A damp-proof course of two coats of hot coal tar was laid on top of the concrete foundation (Jaggard 1921, 16). The external walls were of a similar thickness to cottage n° 4. Channel-shaped-pre-cast-reinforced concrete lintols used over openings and window cills were formed using a cast concrete cill. The internal and external surfaces were finished in the same manner as cottage n° 4.

Construction of the main walls began in the middle of April 1920 and was finished by the end of June 1920. Most of the work was therefore done in the summer months, and the time taken to construct the walls was approximately 10 weeks as opposed to four months for cottage n° 10 and five months for cottage n° 4 (Jaggard 1921, 27).

Cottage n° 10

The main walls of the cottage were built using a variation of the mixture used for cottage no. 4. Portland cement substituted straw. The chalk was broken down after digging so it would pass through a 1fi in. mesh. The chalk was then mixed dry with 1/20th of its weight of Portland cement. No water was added as the chalk already had a moisture content of about 20%. The mixture was thoroughly mixed together by hand on a boarded platform, placed in the shutters, and rammed in a similar manner to cottage n° 4 (Jaggard 1921, 23).

The chalk walls were built on a concrete foundation 1ft. 9in. deep by 1ft. 6in. wide. Four courses of brickwork on top of the foundation and extended approximately 9 in. above finished ground level. Two courses of slates were laid on the

brickwork for a damp-proof course. The brick walling then continued up to height of 3 ft. 6 in. above ground level using English Garden Wall bond with 4fi in. thick inner and outer walls. The 9 in. void between the two walls was filled with well-rammed chalk (Jaggard 1921, 15). External walls were of a similar thickness to cottage no. 4. Strips of 1fi in. mesh wire netting were laid on top of the brick walling at 3 ft. 4 in. vertical intervals in an effort to reinforce the wall. Channel-shaped-compound-wood lintols were used over the openings. Window cills were formed using a splayed brick cill. The internal and external surfaces were finished in the same manner as cottage nº 4.

Construction of the main walls began in early December 1919 and was finished by the end of March 1920 (Jaggard 1921, 25).

The Condition of the Cottages in 1927 & 1945

In the summer of 1927 officers of the Building Research Station based in Garston, Herts carried out inspections on the experimental cottages. In the introduction to the report attention was drawn to the fact that the tenants of the cottages were «of a superior type to the normal occupier of small-holdings and of a superior type to the normal occupiers of cottages of local construction; in several cases the present occupiers are the owners and these owners have accordingly been better cared for than the average cottage of the class». The position of the cottages on high ground some 300 ft. above sea level was also noted and the cottages were exposed to heavy driving rain from all sides; the inspectors suggest that the condition of exposure was considered as very severe (DSIR 36/2197 1927, 1). The report was broken down into the main building components such as foundations & damp-proof courses, external walling, floors and flooring etc. The focus of this paper is the external walling materials and their condition is described using written accounts, photographs and drawings. The main findings of the 1927 report for the three earthen cottages are set out below.

Cottage nº 4 Ratfyn

The 1927 report contains sketches of cracks on this cottage. These sketches were copied, scanned and imported into AutoCAD. The cracking was then traced onto AutoCAD drawings of the elevations. The cracking to the four elevations of the cottages are shown in figure 3. The report states the cracking was quite extensive and the cracks have penetrated below the roughcast in nearly every case. The cracks were reported as being approximately 4 in. deep. The cracking had penetrated through internal plaster in two places: in the wall of the outbuilding on the northeast elevation, and over the lintol on the first floor, although the exact position was not noted. The horizontal string-course below the first floor windows was also cracked on the northeast and southwest elevations. The report states the cause of the cracking as was «due to the shrinkage of the mass of the wall following the evaporation of the water». The report also makes note of some leakages through «straight joints» at the junction of brickwork and chalk and on the northwest elevation where the roof purlins pass through the gable (DSIR 36/2197 1927, 6).

Cottage nº 5. Ratfyn

The report states that the walls of this cottage were «generally more satisfactory than any of the monolithic walls». Some cracking was reported, but was not shown on any of the sketches or photographs. The cracking was considered not as deep or continuous as in the other cottages. There was some concern over the lime slurry peeling away from the main wall with some pieces of the wall attached to it, however, in general, this wall was reported in excellent condition. Note was made in the report that the water content of this mix was lower than the other cottages (DSIR 36/2197 1927, 7).

Cottage nº 10

The report stated that the walls have cracked badly in places, but the cracks were not as numerous as cottage nº 4 Ratfyn. The cracking to the four elevations of the cottages are shown in Figure 4. Cracks were described as starting at the corners of cills and lintels and running vertically. Some of the cracks were described going through to the inside, and, in some cases, the cracks were as much as 1/20 in. wide. The walls of the cottage were described as being dry except where moisture had penetrated through cracks. One damp patch in particular on the southeast wall was caused by bad walling material and a suggestion was made that this was due to bad

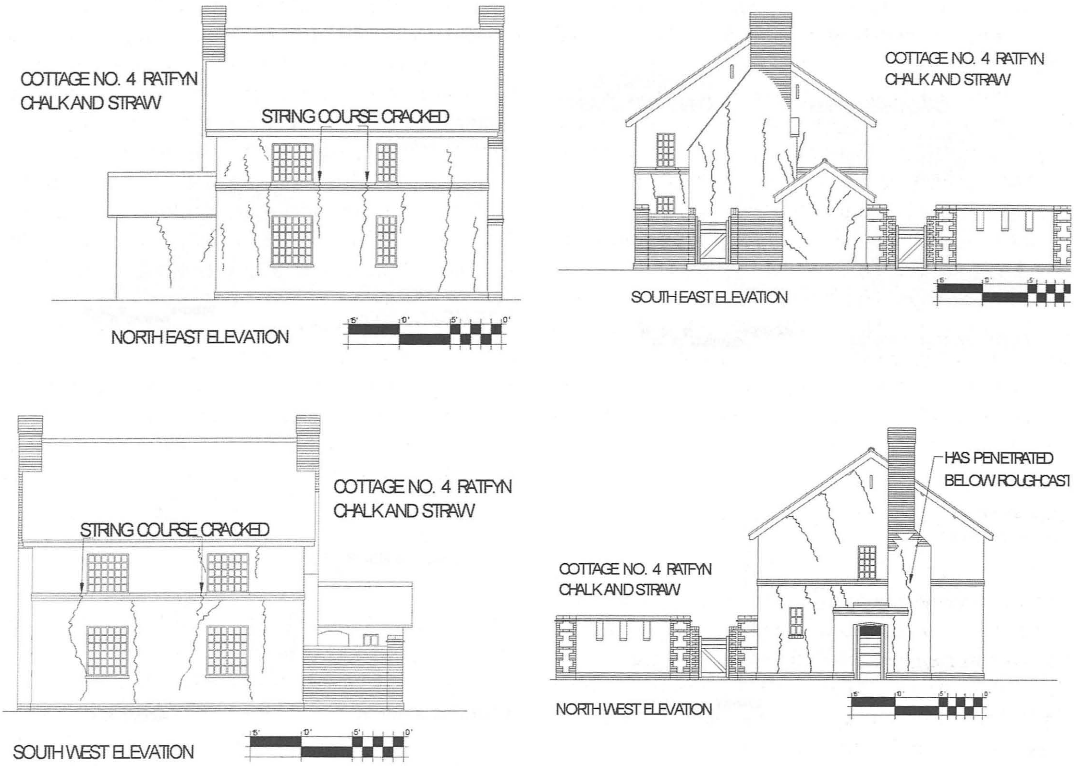

Figure 3
Cracking to cottage n° 4 Ratfyn. Summer 1927

ramming. It was again suggested that the cracking was due to the shrinkage of the walling material and that the wall failed along its weakest planes, between openings. A number of reasons were suggested for the shrinkage. The suitability of wire netting reinforcement to check the cracking was questioned. It was also suggested that a greater bulk change occurred to the wall due to moisture changes in materials used, particularly, the chalk which had a moisture content of approximately 20%. The report suggested that the cracking had finished and any further movement would be taken up by the cracks (DSIR 36/2197 1927, 5–6).

In a general conclusion to the 1927 report there was acceptance of chalk as a walling material. However, there was concern that considerable shrinkage occurs when a chalk wall dries out. The success of cottage n° 5 Ratfyn suggested that using chalk with low water content was the best practice (DSIR 36/2197 1927, 28).

The Cottages in 1945

In 1919, the British Architect, Clough Williams-Ellis, published his masterwork on earth buildings: *Cottage Building in Cob, Pisé, Chalk and Clay.* The book was written, in part, to show how to build rural cottages in materials other than bricks- in short supply after the ending of the First World War. After the ending of the Second World War, a similar situation existed and he republished the book under the title: *Building in Cob, Pisé and Stabilized Earth.* This new edition was expanded and contained a chapter devoted to the experiments at Amesbury under the title *A Successful Experiment.* The Chapter included a description of

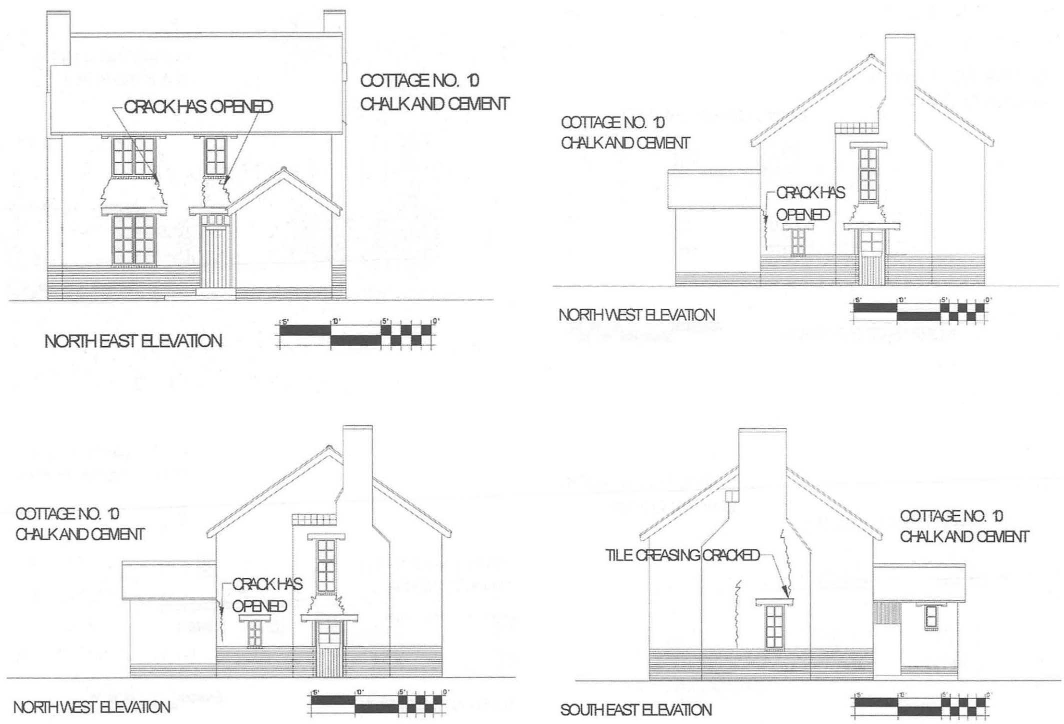

Figure 4
Cracking to cottage n° 10. Summer 1927

the experiments and mentioned examination of the walls in 1927 and 1945. It was assumed that the 1927 examination was the report carried out by the Building Research Station mentioned earlier. The source for the 1945 examination was not made specific in the text. The chapter contained many reproductions of drawings from the Jaggard's original 1921 report, but it also included a number of contemporary photographs showing some of the cottages and specific details.

This additional information about the condition of the three cottages resulted from the 1945 examination. The condition of Cottage n° 4 Ratfyn was reported in a similar condition to that of 1927. It was noted that both the external rendering and the internal plaster was satisfactory. Occasional condensation was noted and this was attributed to the slow warming up of the walls (Williams-Ellis 1999, 129). Cottage n° 5 Ratfyn, considered the best of the monolithic walls in the 1927 report, was again reported as being free of damp and having few cracks after 25 years (Williams-Ellis 1999, 130). Sometime between 1927 and 1945 the external lime wash was replaced with cement roughcast, and, in some places the roughcast came away from the Pisé backing. The cracks present in Cottage n° 10 at the time of the 1927 report were no worse than those reported in 1945. A plate of the exposed outhouse walls of Cottage n° 10 shows, and the text confirms, that the walls have not suffered from attrition, even though they have been exposed for 25 years (Williams-Ellis 1999, 131).

The Cottages in 2002

This author inspected the cottages in the summer of 2002. The objective of the inspections was to identify

any changes that had been made to the properties and to ascertain the current condition of the cottages

Cottage nº 4 Ratfyn (Avonmeads)

This cottage has undergone the most extensive changes since 1945. It is interesting to note this was the only one of the three cottages that was not a listed building. This cottage underwent two major phases of extensions and alterations. In 1979 a two-storey pitched roof addition was added to the northeast elevation. This addition can be seen to the left of the main cottage on figure 5 and the right of the main cottage on figure 6. It contained two garages at the ground floor level and a bedroom, sitting room and bathroom at first floor level. The extension was of cavity masonry construction with a white-painted-cement-render finish. Between 1979 and 1988 a conservatory was added between the cottage and the original outhouse, this can be seen in the center of figure 6. In 1988 a second two-storey-pitched roof extension was added to the southwest elevation. This addition can be seen to the right of the main cottage on figure 5 and the left of the main cottage on figure 6. This extension contains a new study, and dining room and a garage on the ground floor, and three bedrooms and a bathroom on the second floor. This extension was also constructed in cavity masonry with a cement-render finish.

Among the records that the current owner has of the property were the sales particulars of the house

Figure 5
Cottage No. 4 Ratfyn - Northwest Elevation in 2002

Figure 6
Cottage nº 4 Ratfyn. Southeast Elevation in 2002

when the house was purchased in 1978. The particulars include a photograph of the cottage that shows the northwest and northeast elevations. The photograph shows the cottage with a small single-storey flat roof extension on the southwest elevation (this was replaced by the 1988 addition). What was also of interest was the existence of a portion of the experimental wall between the two cottages. It would appear that some of this wall had survived at least until 1978.

An inspection of the earth walling materials was also carried out and the walls were found in a satisfactory condition. It was apparent that the original rendering was replaced. Documents in the possession of the current owner would suggest that the cottage was re-rendered in 1969 with a proprietary textured coating called «Thermo/tex». The owner expressed some concern about the internal plaster adhering to the chalk walling and showed the author an area of damp walling adjacent to the fireplace in the living room. It was apparent that the wall was very damp and further inspection of the wall on the outside revealed that a new patio had bridged the damp proof course.

Cottage nº 5 Ratfyn (Millmead)

This cottage has undergone some changes since 1945. However the changes have generally been at a lesser scale and have not affected the character of the

building to the same extent as additions to cottage nº 4 Ratfyn. This cottage has undergone three phases of additions. In 1972 a single story flat roof addition was added to the southwest elevation. This addition can be seen to the right of the cottage in Figure 7 and to the left of the cottage in Figure 8. This extension was constructed using cavity masonry with a roughcast-cement-render finish. In 1982, the fuel store on the southeast elevation was extended by several feet using cavity masonry to form a utility room. This extension was consumed by a much larger two-storey-pitched-roof extension on this elevation in 1993. This again, was built using cavity masonry with a roughcast finish and contained a sitting room at ground floor level and a dressing room at first floor level. This extension is shown on the right of Figure 8. This addition was much more in keeping with the original cottage in terms of scale and details. Many of the details such as the timber verge details and lintol details, were copied on the addition. This attention to detail and scale was a result of the cottage being

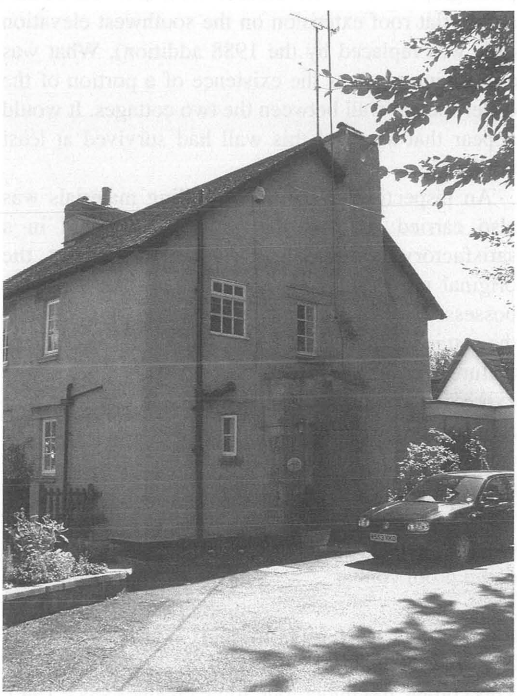

Figure 7
Cottage nº 5 Ratfyn. Northwest Elevation in 2002

designated a grade II listed building on 10 October 1988. Listing a building aims to ensure that any alterations to a building respect the character of the building.

The original chalk pisé walls were re-rendered in 1999. This was done to a high standard and care was taken to identify a suitable render for the property. The new render consisted of a lime- based base-coat, a butter-coat (with roughcast) and a 3-coat lime-wash finish. The local authority who provided approximately 10% of the cost of the work approved this work. An inspection of the cottage revealed it to be in excellent condition and it was noted that the external ground level was kept well below the damp proof course.

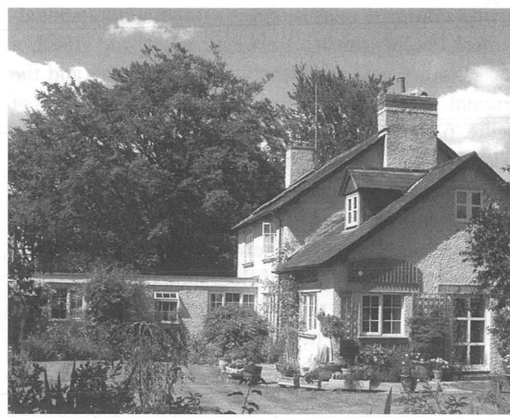

Figure 8
Cottage nº 5 Ratfyn. Southeast Elevation in 2002

Cottage nº 10 (26 Holders Road)

This cottage remains the least unaltered of the three cottages. Figure 9 shows the southwest elevation that remains almost unchanged. The only changes were a small-pitched roof porch that has been added to the northwest elevation, to the right of the cottage in figure 10, and a small flat roof conservatory between the cottage and the fuel store, to the left of the fuel store.

This cottage was also a grade II listed building. An inspection of the cottages walls revealed that they were in a satisfactory condition. The original external finish was still on the cottage as the outlines of some

Figure 9
Cottage n° 10. Southwest Elevation in 2002

Figure 11
Cottage n° 10. Cracking between ground and first floor windows

of the cracks shown in the 1927 report (Fig. 4) were still visible. Figure 11 shows evidence of one of the shrinkage cracks between ground and first floor windows.

CONCLUSION

Various government publications and Williams-Ellis's book have given us a rare opportunity to evaluate the experimental cottages over a period of 80 years. Clough Williams-Ellis claimed the experiment was a success when writing about the cottages in 1945. After inspecting the cottages in the summer of 2002, this author can only concur with Clough Williams-Ellis. All three of the cottages were in a satisfactory condition and two of the cottages have undergone substantial alterations and additions. This was typical of many cottages of this period and it would appear that the experimental nature of the wall construction has not hindered the adaptability of these cottages. The listing of two of these cottages in 1988 should help ensure the conservation of these cottages in the long term.

Figure 10
Cottage n° 10. Northeast Elevation in 2002

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Wood structures in traditional random rubble wall constructions in Cagliari.

Marco Cadinu

Recent research on the residential buildings in Cagliari from late the middle ages to the first part of the 20th century, is providing more information about wall construction as well as more general construction techniques. This research allows to distinguish building methods, the materials used, and the architectural models though the different periods.[1]

One section of the present study, designed to provide a preliminary identification of vertical structures limited to non-monumental buildings, has shed some light on the quality and nature of wall construction. The most recurrent wall typology in the row houses and in the other buildings is composed of random rubble and lean lime. Blocks of tufo stone are used only for door and window frames as well as quoining. This kind of wall is very simple, often made with the help of sustaining structures to contain and press the layers of these materials; they range from around 42–58 centimetres.[2] The static properties of these walls is mediocre because they are heavy and not very resistant to tensile and shearing stress; they were best used for one or two-floor buildings, although sometimes they can be found in four or five-floor buildings.

In the present study we have focused our attention on the taller buildings.[3] We have observed in the wall sections the interesting presence of horizontal juniper wood trunks. In this area of research, wood in the walls had previously been defined as tie-beams used to repair damaged walls. On the contrary, evidence has been found that this wood represents an essential component of the original walls.

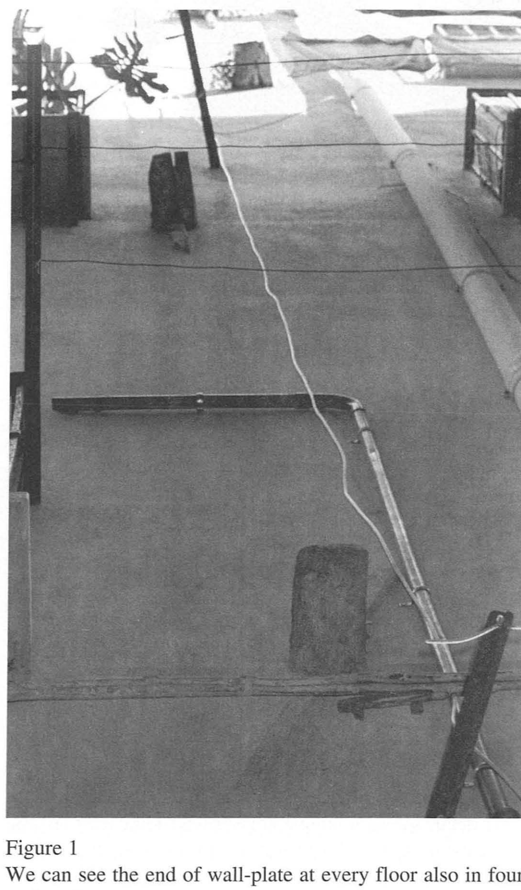


Figure 1
We can see the end of wall-plate at every floor also in four or five-floor buildings

The trunks were attached one to another to form a continuous transversal and longitudinal structure around the walls, making the random rubble construction more resistant and flexible; this hidden structure is also connected to the floor-joists. Moreover, sometimes we have found the presence of vertical wooden elements. The typical scarf-joint is the edge-halfed with a big iron peg. The end of the wall-plate often comes out of the façade, with its wooden punch.

It should be noted that wood structures had not previously been found in Sardinia even if, in these houses, it was common to use timber-frames with brick infill for partition walls. In the poorest buildings wood was often used for lintels of doors and windows. In one instance the use of big wood beams has been documented for building the bearing structure wall of the stairs in a four floor building.

Short juniper woods were used as normal laths for roofs. We found it also in planking (instead of floorboards) or structures of stairs (Cadinu and Zanini, 1996. 53). Other uses were possible, as single pieces inside normal random rubble wall or as roofs

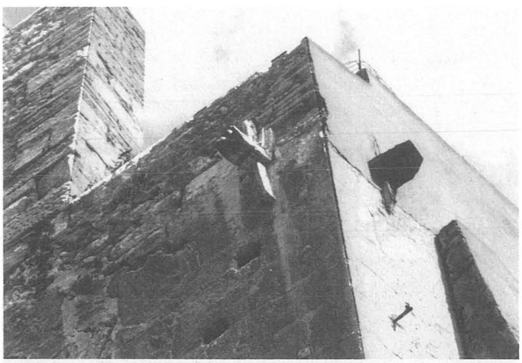


Figure 2
A couple of wooden tie-rod used to repair damaged walls. (Cagliari, Marina, Scalette di Santa Teresa)

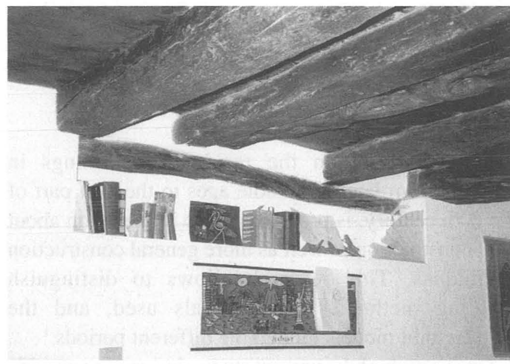


Figure 4
The floor-joist structure is connected with the juniper wood trunks inside the wall

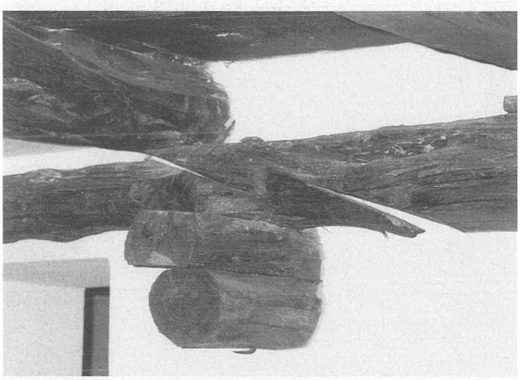


Figure 3
This detail shows a simple edge-halved joint with an iron peg used to connect long series of trunks inside the wall. A similar solution is used for the corners (Cagliari, Marina, Piazza Dettori)

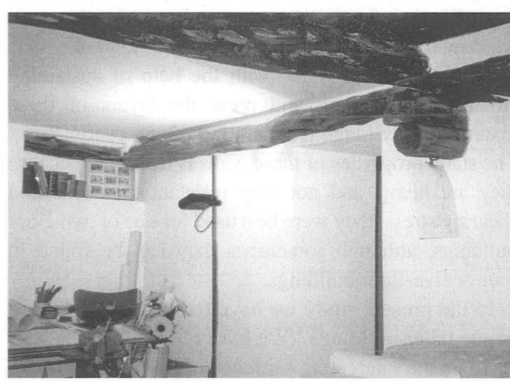
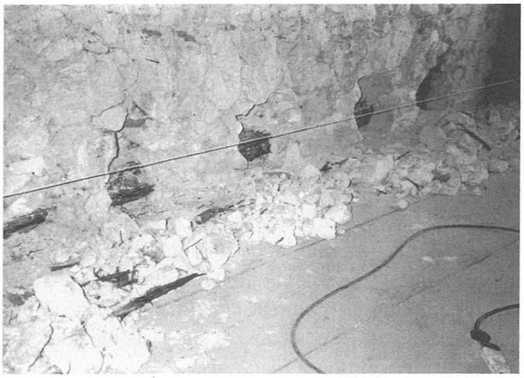
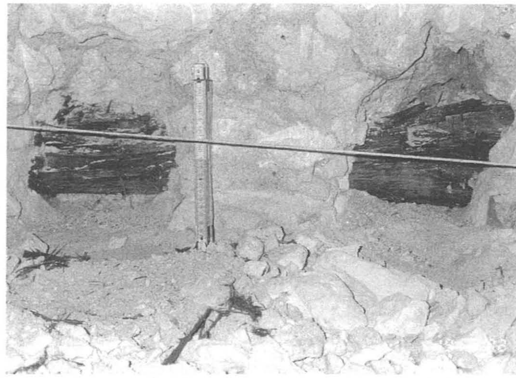


Figure 5
A long wall-plate in juniper attached at the corner of the wall and partially rebuilt outside the wall (a second one was found inside the wall in the same position. Cagliari, Marina, Piazza Dettori)





Figures 6 and 7
A continuous structure of juniper wood trunks found inside a random rubble wall in a four floors building (Cagliari, Castello, Via Lamarmora)

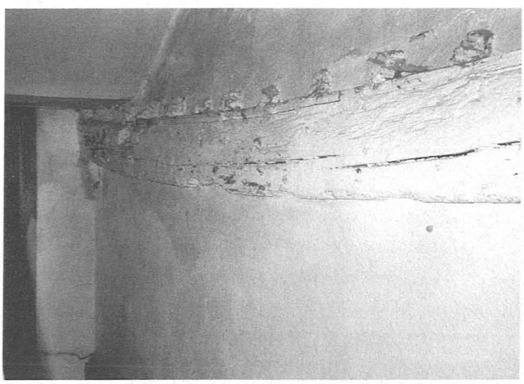


Figure 8
The bearing structure of the stairs is built with wooden beams (Cagliari, Castello, Via Lamarmora)

Figure 9
Irregular vertical wood element in support of a brick partition wall

of a stone drain channel 55 centimetres of clear span.[4] In order to reinforce walls, arches or stairs the use of single juniper woods is permitted in a contract of 1786 were is written they may work «. . . putting inside some juniper wood to best connect them . . . ».[5]

Similar wood structures with wall construction have been previously reported in medieval houses in Parma and Pisa, the latter being the founder of Cagliari in the 13th century. The analysis of some middle-age houses in Pisa have shed some light on the mixed nature of the walls containing a whole wood structure, that it possible to date to the middle-age period of the building.[6] This kind of structure is very simple, no tension brace was used. Up until now no sophisticated techniques of the fachwerk building tradition have been found.

Ancient documents do not report information about wooden houses even if it was observed that such a type was present in some Sardinian towns, particularly in Sassari. A building regulation of 1294–1316 describes the form of the front cutaway on the public street, giving the sizes of projecting joists for each floor. This kind of section is very common in wooden buildings with brick or adobe infill.[7]

It can be hypothesised that the wood building tradition common in the middle ages, and later replaced by stone construction, influenced stone buildings in Sardinia in later centuries. Middle age «Statuti» in Sassari encourage owners to rebuild old houses with new techniques of stone and lime («a petra e a calchina») to improve the general quality of buildings in the town.[8] The presence of skilled workers shows the existence, even in Sardinia, of this old tradition, already common in Italy and Europe in XIII century. Master masons and carpenters (masters *«de muru over de linna»* —walls and wood, masters *«de aschia»*— axe), cited in the building regulations of the «Statuti di Castelsardo» in 1336, article CCXVII, had to receive set daily payments according to time of the year (Cadinu 2001, 178). Also in the «Giudicati» period the presence of such skilled workers like *«maistrus in pedra et in calcina et in ladu et in linna»* (stone and lime, and adobe and wood) has been reported, for example in 1239 (Solmi 1908, 195; 393).

A very interesting document dating back to 1376–77 reports the presence of a big building yard for the renovation and repair of several towers and the port palisade made by a number of stone-cutters «picaperes», carpenters «maestres de axe» or «de fusta», unskilled workers «manobres, bastaix, macip». (Manca 1969). The public contract, registered in detail by the contractor Miguel Ça-Rovira, contains the description of the deliveries, quantities, qualities of each material and also reports the different sizes of wood: «bigues, trapes, sostres, cabirons»; the biggest pieces came from abroad, like 458 big poles «pals» from a place near Rome or big beams «fusts» from the cost of Ogliastra, east of Sardinia.[9]

We think that some of the old wooden building tradition survived and perhaps affected the most modern building techniques. Close contacts with Spain from the 14th to the 18th centuries clearly favoured the development of these techniques. Simple timber frames with adobe infill were indeed very common in the popular Spanish architecture.[10]

Juniper wood

Juniper wood, used also for roof trusses and floor-joists up until the beginning of the twentieth century, can be found nowadays only in bushes. In traditional buildings juniper has been found in trunk ranging from 3,80 to 5,40 meters of length, with diameters between 16 and 32 centimetres. Nowadays in nature trunks of these dimensions are quite rare and can be found only in parks and protected areas.

According to our assessment of pre-industrial buildings in the area of Cagliari, at least 250,000 trunks can be found. It should be noted that these buildings have been constructed for the most part between the XVII and the beginning of the XX. This number is remarkable not only from the point of view of the historical assessment of human impact on the environment but also because it suggests the need for alternative resources in materials.

Although very irregular in shape, juniper possesses extraordinary characteristic in terms of strength and durability. Its fibres, long and compact, can easily wrap around the trunk up until 1.5–2 meters. In spite of the

Figure 10
A timber-frame with brick infill

Figure 11
The front view of a square hole built to drain waters by a bucket directly from the highest floors (Cagliari, Marina, Piazza Dettori)

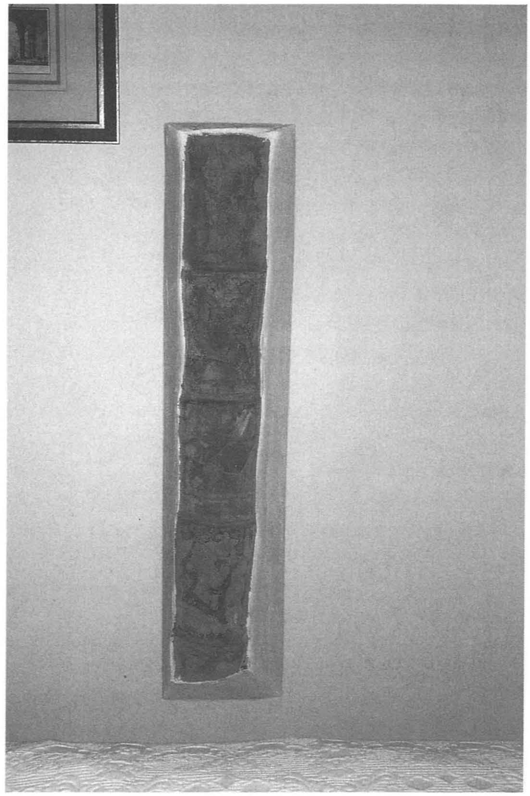


Figure 12
A drain pipe made with piled cotto elements inside the walls. This pipes drained the water to a underground family tank that has been dug under each building (Cagliari, Marina, Piazza Dettori)

irregular structure that often causes performance decrements, the average resistance of juniper wood can be estimated to be more or less double to the resistance of a pine wood. The characteristic of its resin gives the juniper wood its typical scent and protects it from fungi, woodworm and white ant. Only under severe and persistent conditions of humidity a partial deterioration can be found. Thus, juniper durability appears to be excellent.[11]

Inside the walls.

In the traditional houses in Cagliari other elements inside the house walls make the analyses more complex, particularly in the context of restructuring and renovation interventions.

Rain waters were generally collected by drain pipes made with piled cotto elements inside the walls. This pipes drained the water to a underground family tank that had been dug under each building. Waters were drained directly from the highest floors inside the wall through a hole as large as a bucket.

From the kitchen a flue inside the wall typically took the smoke to the top of the building but often only to the front. The last example of this system, forbidden only after the building regulations in 1840, can be observed in an old house built after the end of the XVII century in Cagliari, at the corner of Santa Croce church.[12] The squared holes inside the walls

Figure 13
A different kind of drain pipe made with piled cotto elements inside his own squared hole built inside the wall (Cagliari, Castello, Via Lamarmora)

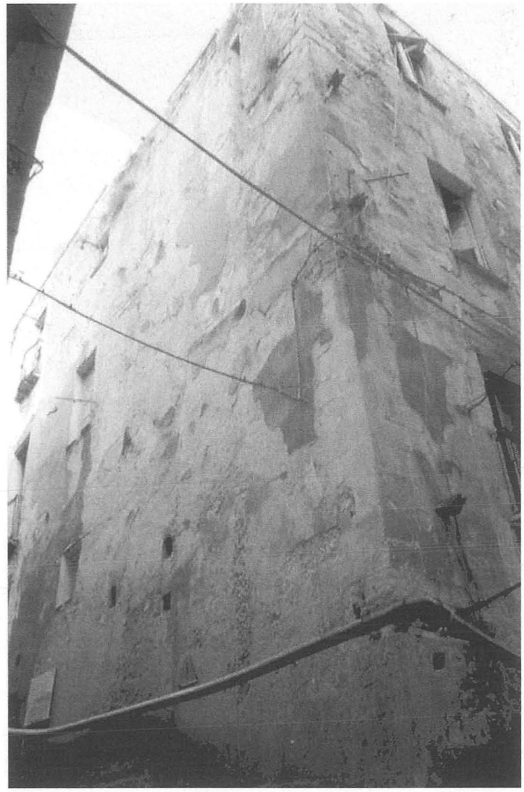


Figure 14
In the front views of this house, built after the end of the XVII century in Cagliari at the corner of Santa Croce church, we can see the end of wall-plate at every floor and on the left, from kitchens, the old front chimneys. This system was forbidden after the building regulations of XIX century

were often closed on the internal side by brick infill built between little beams.

NOTES

1. This research, conducted At the Dipartimento di Architettura dell'Università degli Studi di Cagliari, has the goal to compile an handbook for the restoration of Cagliari, with the metods of the «Manuale del Recupero», (Marconi et al. 1989). In the city of Cagliari many historical building regulations made the buildings different from the rest of its countryside area, where the most tecniques use the adobe wall. A previous study in this area (Tradition of construction within the national territory: continuity and evolution of building techniques for the environmental safeguard of settlement context) has been conducted by the Prof. Antonello Sanna and Prof. Adolfo Cesare Dell'Acqua at the Universities of Cagliari and Bologna, respectively.
2. Some researches in the Archivio Comunale di Cagliari permitted to find some description of this kind of wall, like the one in a document written in a contract of 1786 for a new part of the City's Palazzo Civico: «La costruzione della muraglia verrà formata con cantoni di pietra tramezzaria o mattoni per riguardo alla facciata angoli, spalle volte, fasce l'interno però della muraglia si potrà costruire in pietra ordinaria . . . e nel scagliare la suddetta muraglia dovranno battersi con martello a forza le pietre ben nuotate in calcina, e serrate.», see Melis (1997–98. 93–94).

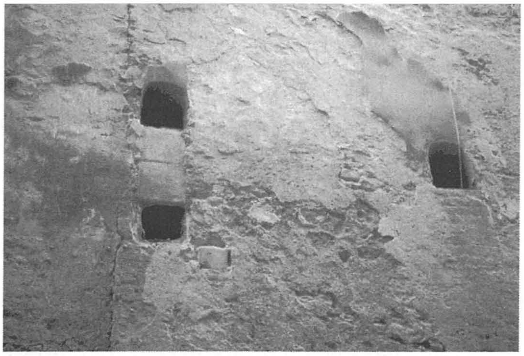


Figure 15
Two old front chimneys with their outward sides brick infill (on the right) or cotto tile (on the left)

3. Some building yards of restoration of old houses in Cagliari Castello and Marina, directed by myself, have provided exemplars presented in this meeting.
4. The possibility of finding in nature big juniper curved trunks allowed builders in the past to use them for two pitches roofs. In the past they where described as false roof trusses, even if they are simple beams.
5. « . . . mettendovi qualche legno di ginepro per ligarli maggiormente . . . », see in Melis (1997–98. 96).
6. (Redi 1996, 89–100). In Parma some studies have reported the presence of some middle age wooden houses with their survived wood parts inside the walls, see Doglioni, Merli and Storchi (1987, 505–516); regarding houses in Pisa see Redi (1989, 107–117; 146–151) and the old good classic Lupi (1901).
7. From the building regulations of Sassari, Libro I, XXXVIII: « . . . et supra alcuna via qui aet esser de palmos XV, over minus, non se fathat solaiu in alcuna domo posta testa ad ecussa via, su quale solaiu essat foras dessu muru su plus palmos iij, et ciò in su primu solaiu, et ecussu solaiu gasi postu testa a bia siat altu palmos XIV. Et de cussa altithia siat [su tectu] over grunda de çascatuna domo posta testa a via et non essat cussa grunda over tectu foras dessu muru plus de cussu qui est naratu dessu solaiu, et si alcunu aet alçare sa domo sua, qui ait esser testa a via publica in altithia de duos solaios, su secundu solaiu non essat foras de su muru de cussa domo vltra palmos IV. Et si alcunu aet bollet alçare sa domo sua dave duos solaios in susu, in çascatunu solaiu pothat essire palmos V.».

A drawing of the cutaway of this houses with projecting joists for each floor was compared with others coming from the fackwerk areas (Cadinu 2001, 116). Regarding projecting joists in fachwerk buildings see Walbe (1979. 9).

8. Old houses were built in stone and adobe, «de petra et de lutu», see again in Cadinu (2001, 152).
9. (Manca 1969). More observations about names and meanings in Cadinu (2001, 182–190), «Glossario dei termini urbanistici ed edilizi».
10. Examples from Manzaneda de Torío (León) and Calahorra de Boedo (Palencia) look very interesting and show closing methods common to distant areas that will be useful to study in the future. See Ponga Mayo and Rodríguez Rodríguez (2000. 135; 141).
11. Neither laboratory tests about the static properties of juniper woods (juniperus phoenicea but also juniperus oxycedrus oxycedrus and juniperus oxycedrus macrocarpa) have ever been conducted, nor tests for the compilation of a dendrocronological list; we have planned a research project on this topic.
12. The neighbours at the highest floors had to allow the building of a new chimney to the top of the building, see in «Archivio Comunale di Cagliari, Atti Amministrativi e Governativi, Regie Patenti, Torino 11–4–1840, Titolo VI, art. 47», also in Cadinu (1996. 97).

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Orthographic projection and true size in Spanish stonecutting manuscripts

José Calvo López

Stonecutting treatises, from the Renaissance to the Enlightenment, classify dressing methods in two broad areas. Both are named mainly after masons' tools: the square —*équarrissement*, *escuadría*, *dérobement, robos*— and the template —*panneaux, plantas*. However, the significance of these categories lies mostly in the geometrical notions they stand for; the square is applied when tracings make use of the orthographic projection, usually double, whereas the template is used where geometrical constructions provide true-size representations of the sides of the dressed stones. Thus, both methods are related to key concepts in Descriptive Geometry, the double orthographic projection for the square and developments and *rabattements* for the templates. This is not surprising, if we take into account that Gaspard Monge, the founder of Descriptive Geometry, was Professor of Theory of Stonecutting in the *Ecole de Génie* at Mézieres (Monge 1799, 4; Loria 1921:84–91; Taton 1954, 17–20; Sakarovitch 1992, 530–536; Sakarovitch 1995, 208–210; Sakarovitch 1998, 218–227; Rabasa 2000, 241).

The picture is not so clear as it appears in the treatises, however. It has been pointed out (Rabasa 2000, 158–160) that modern stonecutting mixes both methods, and that Frézier (1737, 2:14; 2:115–116), and his Spanish follower Bails (1779, 433–437) propose the dressing of some pieces by a method known as *demi-équarrissement* or *media escuadría*.

We shall deal in this paper with an early example of these hybrid methods. In *Cerramientos y trazas de montea,* a manuscript written by Ginés Martínez de Aranda around 1600, we can find stonecutting problems solved by means of pure squaring or full templates, the two canonical methods of mainstream treatises. Nevertheless, Aranda also explains the use of a two-side adjustable template, the *saltarregla* or *sauterelle*, combined with squaring; and what is more remarkable, the combination of squaring and full templates. First, we shall discuss stonecutting operations based only on orthographic projections or on true size, taking into account not only Aranda's manuscript, but also other Spanish works and, when the need arises, Philibert de L'Orme's *Premier tome de l'Architecture*; afterwards, we shall study the hybrid methods that appear, here and there, in Aranda's manuscript.

Orthographic projections

Squaring

Martínez de Aranda (1600, 113–114) includes a short, but fairly clear, description of the squaring method:

> Supongo que la figura *A* es el bolsor que quieres entrar en cuadrado y el dicho cuadrado son los cuatro ángulos *a b c d* con el cual dicho cuadrado cogerás los extremos del dicho bolsor y robándolo por el lecho alto con el robo **a** y por el tardos con el robo *b* y por la cara con el robo **c** y por el lecho bajo con el robo *d* y pasando los dichos robos

de una testa a otra quedará formado el dicho bolsor como parece en la figura *B*.

Therefore, squaring involves obtaining the orthogonal projections of the faces of the voussoir; inscribing the voussoir in an imaginary rectangular prism; cutting roughly a block with the dimensions of the prism; and taking off a number of wedges from the block with the aid of the projections, until the voussoir reaches its definitive form. Aranda does not employ the word *escuadría*, as Benito Bails (1779: 428, 430, 433) will do later; instead, he uses *robo* and *robar*, from the French *dérober*, meaning to take off or divest. However, the square allows the stonemason to transfer the wedges from one face of the block to the other; when one arm of the square rests on a flat side of the voussoir, the other arm materializes lines that are perpendicular to the flat side. In this way, the square can generate planes at right angles to the faces of the voussoir, such as the joint planes, or cylinders whose axis are perpendicular to the planes of the faces, such as the intrados surface.

This procedure has a clear geometric meaning. Architectural drawing handbooks, and even Descriptive Geometry texts, refer to the process of passing a line through a point and finding the intersection of the line with a plane, as projection. Thus, the projection of a point is a point, the projection of a straight line is a straight line, and the projection of a curve is generally a curve. This is what Aranda did when he traced the face of an imaginary voussoir. Nevertheless, in the more rigorous vocabulary of Projective Geometry, that process is a two-step operation of projection and section, and strictly speaking, the projection is only the first phase. Hence, the projection of a point is a line, the projection of a line is a plane, and the projection of a curve is, generally, a surface; thus, the intrados surface is generated as a orthogonal projection of the face arc by means of the square.

Aranda's description is purely didactic, since it involves the voussoir of a semicircular arch. In this case, all voussoirs are identical to the keystone; it is advisable to dress all stones using the projections of the keystone in order to minimize the volume of the enclosing block. Doing so, there is little difference between squaring and *panneaux*, for the horizontal projection of the intrados is the same as the intrados template.

Alonso de Vandelvira (1580, 24 v.) gives a fairly detailed description of the squaring method applied to a real stonecutting problem in the *Arco en torre cavado y redondo por robos*, that is, an arch opened in a curved wall, carved by squaring. His aim is also clearly didactic. He has explained before a more economical solution by *panneaux*, but he describes the squaring alternative «por que también sea lumbre para entender otras trazas que no se pueden hacer si no es por robos»; that is, to cast light on other problems that can be solved only by squaring. It is interesting to note that when discussing the *panneaux* solution, Vandelvira is afraid that the reader will not be quite convinced. To leave any doubt aside, he advises to make a model of the arch by squaring: «si lo quisieres probar contrahaz un arco de éstos por robos, como te enseñaré adelante, y luego planta estas plantas y harás la prueba ser éstas ciertas». Thus, Vandelvira accepts as an empiric proof the simple squaring method, rather than to the complex *rabattements* of the *panneaux* method.

According to Vandelvira, «después de haber trazado su arco y torre cavado y echados sus plomos, pondrás las piezas en cuadrados desde las tardosas a las mochetas, así en el arco como en el grueso de la pared del torre cavado». That is, the squaring method requires a tracing, usually made in full scale on a floor or a wall; but the tracing is relatively simple, compared with the more elaborate operations necessary to employ the *panneaux* method. It will be sufficient to construct a plan and an elevation of the arch, to divide it in voussoirs, and to trace an enclosing rectangle around each voussoir, both in plan and in elevation.

The first operation of the stonecutting process is to carve a block with the dimensions of the enclosing rectangles. In the next step, the stonemason should take off four or five wedges from this basic block to give the voussoir its final shape. There are small differences with the procedure explained by Aranda. In an ordinary voussoir, the mason should take off the wedges of both joints, that of the intrados, and two additional wedges, corresponding to both curved faces. In the first voussoir or *sommier*, there is no need to take off a wedge for the low joint, given that it is horizontal. As usual in Spanish stonecutting, nothing is said about the extrados, either in the ordinary voussoir or in the *sommier*.

However, it is striking that Vandelvira advises to carve the curved faces in the first place: «lo largo su

torre cavado y redondo . . . lo cual se ha de robar primero que nada». Doing so, the stonemason can dress those faces with the aid of a square leaning in the horizontal upper plane of the block, since both curved fronts are cylinders with vertical generatrices. It is not easy to understand how the stonemason will carve both joints and the intrados. According to Vandelvira, the intrados should be dressed «con la cercha del arco echándola por cuadrado»; that is, with a one-sided template or *cerce*, keeping it squared. Looking at the tracing, it becomes evident that the *cercha* represents the orthographic projection of the arch on a vertical plane perpendicular to the axis of the arch. Since Vandelvira makes no attempt to develop the arch, then «echándola de cuadrado» means keeping it parallel to this vertical plane. However, this is not easy, for this plane is parallel to the planes of the front and rear sides of the block; and these sides were suppressed when the curved faces were dressed before the intrados, following Vandelvira's explicit instructions.

In my opinion, the simplest solution to this riddle lies in Aranda's remark that the *robos* should be brought from one face to the other. In our case, the stonemason must carve the basic block and mark on it the shape of the face, obtained by orthographic projection. After that, the mason can make four straight linear courses or *tiradas* joining the vertices of the orthographic projections of the arch in both faces, still planar, before dressing the curved fronts. After carving the curved fronts, the stonemason can easily dress the joints with the aid of a ruler leaning in two *tiradas*, since all straight lines that intersect two parallel lines are in the same plane. More care is needed in the dressing of the intrados; that is why Vandelvira states that the *cercha* should be kept *de cuadrado*, that is, orthogonal to both *tiradas*. Doing so, the *cercha* is parallel to the projecting plane, since these *tiradas* are orthogonal to the vertical plane of projection, and two planes that are orthogonal to a straight line are parallel to each other; in this way, the movement of the *cercha* generates the intrados cylinder.

Projected templates

Of course, the dimensions of the block can be transferred to the stone by means of a gauge or any measuring instrument, and there is no need to use templates in this step. If the definitive shape of the voussoir is fairly simple, it can be brought to the stone in the same way; this seems to be the case in Vandelvira's explanation. However, stonemasons employed templates to transfer complex voussoir faces, even in the context of the squaring method (Frézier 1737, 2:12–13; 2:108–109; Pérouse de Montclos 1982, 90; Palacios 1986, 102). It is interesting to note that for Martínez de Aranda the templates used in the *panneaux* method are *plantas al justo*, which can be translated as «exact templates»; but when templates are used in connection with the squaring method, the operation is called *plantar de cuadrado*, a word that means *orthogonal* in sixteenth-century masons' jargon, and comes from the same root as *escuadra, escuadría, équerre, équarrissement* and *square* (Martínez de Aranda 1600, 7, 12, 85).

True size

Templates

Vandelvira and Aranda explain how to construct true size templates in the majority of the *trazas* of their respective manuscripts. Leaving aside flexible templates, and those that try to represent a warped quadrilateral, these methods are based either on an antecedent of the *rabattements* of Descriptive Geometry or on triangulation. Both authors give fairly similar descriptions of the tracing of the *plantas* in the *Arco viaje contra viaje por testa*, or simply *Viaje por testa*, a skew arch (Vandelvira 1580, 19 v.; Martínez de Aranda 1600, 9–11). After tracing the arch in plan, the mason should construct an elevation with the projection plane orthogonal to the intrados joints. Since the arch is biased, the picture plane cannot be parallel to the arch face. To construct the intrados template, the stonemason should rotate the intrados of a voussoir around the lower intrados joint; of course, this joint will not move in the rotation. Since the upper intrados joint is parallel to the lower joint, it will stay parallel after rotation; besides, he can read the distance between both intrados joints from the elevation, for both are perpendicular to the projection plane and are projected as points; in this way, the stonemason can trace the upper intrados joint. To find the ends of this joint, he can take into

account that both move in a plane that is perpendicular to the axis of rotation; the horizontal projection of this plane will be a line that passes by the projection of the end of the joint and is orthogonal to the lower intrados joint; the end of the joint shall be where this line intersects the upper intrados joint. In this way, the mason can trace the intrados template, knowing its four vertices.

However, this method makes the face arc elliptical and the cross-section of the arch semicircular. For aesthetic reasons, from the 16th to the 18th centuries, architects and stonemasons usually preferred arches with semicircular faces and elliptical cross-sections (Frézier 1737, 1:279; Rabasa 1994, 147; Rabasa 2000, 304). De L'Orme struggled to explain this solution in an obscure passage (1567, 67 v. – 69 r.) that suggests the problem was addressed by means of squaring.

Vandelvira (1580, 28 r.) tried to solve the problem constructing an elevation so that the projection plane is parallel to the face plane; this allows him to trace easily the semicircular face arc. The trade-off was that the intrados joints were not orthogonal to the projection plane, and that prevented Vandelvira from using the simple rotation technique he had used in the *Viaje por testa*. To overcome this difficulty, he resorted to a triangulation technique, constructing the intrados template from the lower intrados joint, that is horizontal and, hence, represented in true size in the horizontal projection. To obtain an end of the upper intrados joint, he constructs a right triangle whose catheti are the horizontal projection of the diagonal of the intrados template and the difference in heights of both ends of the diagonal; of course, the hypotenuse is the length of the diagonal in true size. Vandelvira traces then two arcs, one with centre in one end of the lower intrados joint and radius equal to the length of the diagonal and other with centre in the other end of the lower intrados joint and radius equal to the length of the chord of the face arc. The intersection of both arcs gives one end of the upper intrados joint. The other end of the upper intrados joint can be placed using the same construction; that allows tracing the intrados template. Though this construction is clever, it is also slow, recursive and prone to accumulate errors. Ginés Martínez de Aranda, Alonso de Guardia and, probably, Cristóbal de Rojas use a different construction based on rotations to solve the problem, but we shall deal with it later (Cristóbal de Rojas 1598, 99 v.; Martínez de Aranda 1600, 16; Alonso de Guardia 1600, 80 v.; Calvo 1998, 69–70).

None of the Spanish manuscripts includes a full explicit description of the stonecutting process using these templates. According to Philibert De L'Orme (1567, 99 r.):

> pour les paneaux de ioincts, paneaux de teste, & aussi paneaux de doile par le dessus, gardez vous bien de les trasser pour coupper la pierre du premier coup, car vous la gasteriez, & ne pourroit plus seruir. Il fault doncques oster vn peu de d'vn des ioincts, & puis vn peu du costé de la teste, semblablement du costé de la doile de dessus, . . . & non point tout à vn coup, mais couppant si dextrement le tout que vous puissiez armer vostre pierre de paneaux tout autour qui se rapportent iustement & se touchent l'vn l'autre par toutes leurs extremitez, tant que par les ioincts que par les doiles & par le deuant, ou est le paneau de teste

Normally, the use of pure templates is not as difficult as it appears from De L'Orme's colourful description. Philibert is talking about a *trompe*, a cut that involves a specific difficulty in the acute vertex of the triangular shape of the intrados. Applying his method to the voussoir of an arch or vault, an ordinary stonemason can dress a flat face in the intrados and inscribe the shape of the intrados template on it. After that, he can start gradually taking stone off from two adjacent sides of the voussoir, say the front and the upper joint, shaping two planes that pass by the intrados joint and by the chord of the face arc. It is important to check the result at intervals by means of the corresponding joint and face templates, as they approach the position in which both templates assemble with each other and with the intrados template. When this point is reached, the face joint is fixed in space, and the planes of the face and the joint are also fixed; the stonemason can dress both easily with the help of the ruler.

To our ears, this procedure sounds inneccesarily complex and dangerously empiric. In the 18th century, Frézier (1737, I:372–374), addressed the problem in a more efficient way, using the dihedral angle between the intrados and joint planes; that is, the angle between the intersections of both planes with a third plane that is orthogonal to their common intersection, in this case the intrados joint. Apparently, this idea is too abstract for De L'Orme or the Spanish manuscripts of the 16th century, for none of them mentions it. However, the subsequent steps in the

dressing process we are describing are much simpler. Once the planes of the face and the joint are dressed, the stonemason can in turn inscribe on them the corresponding templates. After this, the mason can dress the plane of the lower joint with the aid of a ruler leaning in the intrados joint and the face joint, inscribe the joint template on it, and dress the rear face making the ruler rest on both face joints and the chord of the face arc.

As we have said before, no Spanish manuscript or treatise describes this method explicitly. However, Martínez de Aranda explains the construction of intrados and joint templates in almost each *traza* of his manuscript. Face templates are usually present, either included in the elevation belonging to the tracing, or either by means of the construction of the *cimbria*, that is, a set of face templates. This suggests strongly that Aranda considered this procedure as the canonical method in stonecutting. Perhaps for this reason he never explains it explicitly, while he remarks the details that diverge from this paradigm, as we shall see below.

Templates and saltarreglas

Notwithstanding that, some evidence suggests that the most widely used dressing method in 16th Spain was a simpler one. Alonso de Vandelvira usually explains the construction of the intrados templates, calling them simply *plantas*, but not the joint templates, the *plantas por lecho* of Martínez de Aranda. Instead, he constructs usually *saltarreglas*, that is, lines that represent the face joint and allow him to measure the angle between the intrados joint and the face joint; the *saltarregla* takes its name from the protractor using by the stonemason to transfer these angles, from the French *sauterelle*. Some interesting details make clear that the ensemble of these *saltarreglas* and the intrados joint plays the role of a simplified joint template. In his *Viaje por testa*, Vandelvira (1580, 19 v.) explains the construction of *plantas* and *saltarreglas*, adding that «si quisieres echar molduras has de extender los moldes en las saltarreglas». The result is the most complete joint template one can think of, representing not only its four edges, but also a highly detailed section of the moldings. On the other hand, in many *trazas*, Aranda (1600, 16, 19, 25, etc.) represents the three edges of the joint template corresponding to the intrados joint and both face joints in solid line, while he renders the extrados joint in dashed lines. This graphic treatment seems to let the reader choose between applying the intrados and face joints as *saltarreglas*, and using four joints as a full template.

Vandelvira gives also few hints about the way *plantas* and *saltarreglas* are used. However, the consideration of the *saltarregla* as a simplified template suggests a variation of De L'Orme's method. After dressing the plane of the intrados side of the voussoir and marking on it the intrados template, the stonemason can gradually take material from the face side of the voussoir, until the face template and the protractor, opened in the angle marked by the *saltarregla*, assemble in the face joint. This method is less cumbersome than the use of joint and face templates, for the protractor can be used even if the joint is not dressed yet. Once the first face joint is fixed in space, stonecutting can go on as we have explained before. Inversely, Alonso de Guardia (1600, 82 v.) explains the dressing of a voussoir of the *Arco abocinado* using an intrados template, a joint template and the *baivel* or bevel of the arch.

Hybrid methods

Orthographic projection and rotation

Squaring is not economical, neither in material nor in labour (De L'Orme 1567, 73 v.). Renaissance stonemasons tried to find methods, still based in orthogonal projections, that would reduce this loss. One of these consists in inscribing the voussoir, not in a rectangular prism with horizontal and vertical faces, but in a mixtilinear prism that circundates closely the projection of a voussoir in a vertical plane.

In the *Arco capialzado por robos*, Martínez de Aranda (1600, 40–41) explains the carving of the voussoirs of an arch with a semicircular arch in the front face and a segmental arch in the rear. As in other occasions, prior to the dressing of the voussoirs, the stonemason is to make a full-size tracing. This tracing is fairly simple, for it involves the construction of the plan of the arch; the elevation, with the semicircular and segmental arches; and the division of the arch in voussoirs, by means of a set of planes that pass by the axis of the segmental arch. It is interesting to note that

Aranda does not represent the plan of the intrados joints, for he will not use their horizontal projection in the carving process.

The lines crossing the intrados in the plan are not projections of the intrados joints, but rotations of them, constructed with the aim of measuring the angle between front joints and intrados joints; this rotations of the intrados joints are called *saltarreglas* by Aranda. It is important to take into account that Vandelvira gives that name to face joints, not intrados joints, traced also to measure the same angle. Thus, the word *saltarregla* seems to be associated to the angle between both joints; that is not surprising, for the *saltarregla* is in the first place the stonemasons' protractor or *sauterelle*.

To construct the *saltarregla*, Martínez de Aranda rotates the line 1 5 around a horizontal line orthogonal to the projection plane, that is, the plane of the arch face. In this way, he brings the line 1 5 to a horizontal plane, to measure its angle with the front joint. In this rotation, the point 1 does not move, since it belongs to the axis of rotation. The point 5 will move in the plane of the arch face, since this plane is perpendicular to the axis of rotation. Besides, the distance from 5 to the axis of rotation can be read from the elevation, since the axis of rotation is perpendicular to the plane

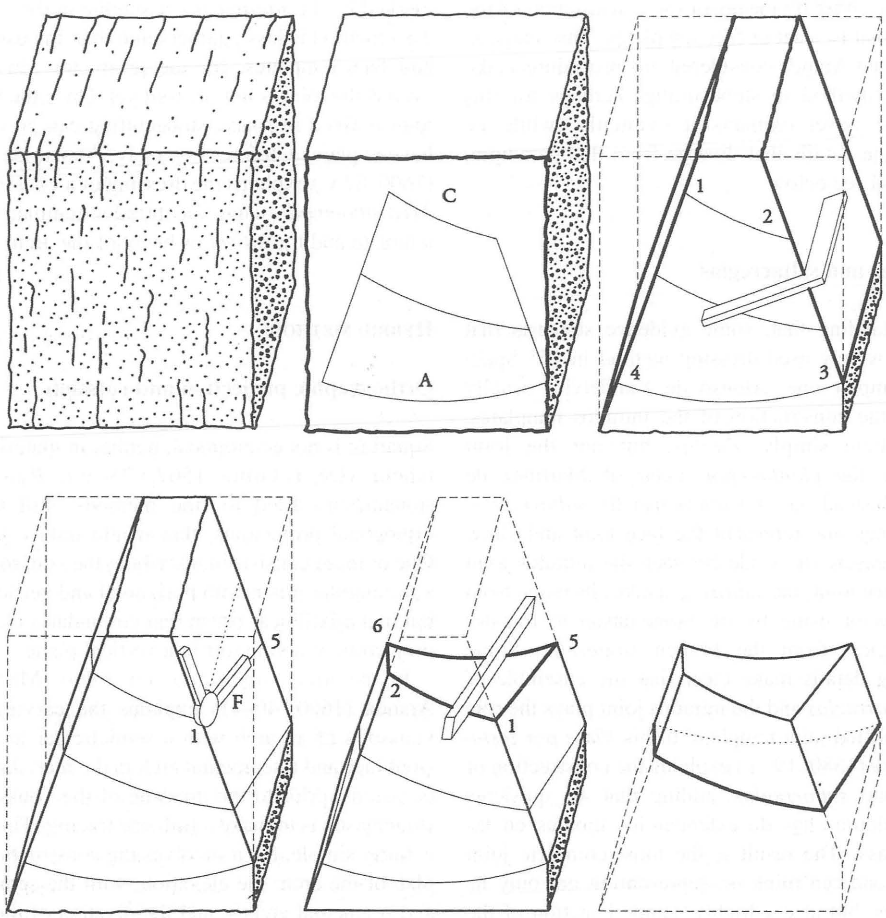

Figure 1
Dressing of a voussoir of the *Arco capialzado por robos*

of projection and therefore it is projected as point 1. Transferring this distance to the plan from the axis of rotation, Aranda can place the point 5 after rotation; joining it with 1, he can construct F, which represents 1 5 after rotation. As for the face joint, it will move in the face plane; so it will be represented by the horizontal projection of the arch face. Repeating the operation for the other side of the voussoir, he can construct G, representing 2 6 after rotation.

Once the tracing is done, the stonecutting process is fairly simple, and much more economic in labour and material than ordinary squaring. The stonemason starts by carving a mixtilinear block with the base in the shape of 1 2 3 4, and height equal to the distance between B and D. After that, he can mark the angle between the intrados and the front joints in the joint side of the block, with the aid of the *saltarregla* or protractor. Repeating the operation on the other side, he is able to place the points 1 2 5 6, that define the four vertices of the intrados face of the voussoir, and to take material from the block until the intrados face is correctly dressed. In the *Arco capialzado viaje por cara,* which is a biased variation of this arch, Martínez de Aranda (1600, 46) suggests the use of a ruler to check the dressing of the intrados, stating that the ruler should create a non-developpable ruled surface: «de unas testas a otras las labrarás a regla plantando la regla de cuadrado que vengan a quedar por las caras engauchidos«.

Orthographic projection and true size

Using the same technique in more complex pieces, Aranda arrives gradually at the combined use of templates and squaring. In the *Arco por arista en la cara,* Martínez de Aranda (1600, 46–47) addresses the problem of an arch formed by two *arcos capialzados* joined back to back; that is, an arch with a «V» section, with two semicircular faces and an *arista* or groin in the form of a segmental arc, on an vertical plane parallel to the planes of the two faces. As in the preceding example, Aranda starts by making a simple tracing, that represents the plan of the arch with both faces and the intermediate groin; the elevation with the semicircular fronts and the segmental *arista*; and the division of the arch in voussoirs. To construct a schematic joint template of each voussoir, he rotates both segments of the V section around a line that is orthogonal to the faces of the arch and passes through the intersection of the *arista* and the plane of the joint. This point is on the rotation axis and will not move. Point 5 and its counterpart in the other face will move in the face planes; he can take their distances to the axis of rotation from the elevation and bring them to the plan; joining the two points with 1, he can trace a simplified joint template in true size. Of course, this is no more than a duplication of the method Martínez de Aranda had used in *Arco capialzado por robos,* but instead of a single *saltarregla*, the result is a template representing the «V» cross-section.

The description of the stonecutting operations hints strongly that the method is transitional between squaring and templates. According to Aranda, «la labraras primero de cuadrado con la forma que tuviere entre los cuatro puntos 1 2 3 4 que tenga de grueso lo que tuviere de ancho la planta del dicho arco»; that is, the stonemason should carve a mixtilinear prism, with the width of the arch and the shape defined in the elevation by the projections of the extrados, the groin, and the two joints. After this, he should take off the wedges that are below the ruled surfaces that pass by a face arc and the groin; to do so, Aranda advises to mark in both joint sides of the block the shape of the «V» cross-section of the arch. This section is not a single line, as in the *Arco capialzado*; hence, Martinez de Aranda designates it as *planta por lecho*, or joint template, while using the traditional word for squaring: «la robarás por entrambas testas con el robo que parece entre los números 1 2 5 6 que venga a quedar después de robada por el lecho bajo con la forma que tuviere la planta por lecho F y por el lecho alto quedará con la forma que tuviere la planta por lecho G».

True size and orthographic projection

In the *Arco por arista en la cara*, the stonecutting procedure is based mainly on orthographic projections, but Aranda uses a true size template as an auxiliary device. Inversely, in the second *Arco en viaje por cara y por plantas* of the *Cerramientos and trazas de montea*, the dressing of the voussoirs is carried on with the aid of full size templates, but Martínez de Aranda (1600, 15–16) advises to use the

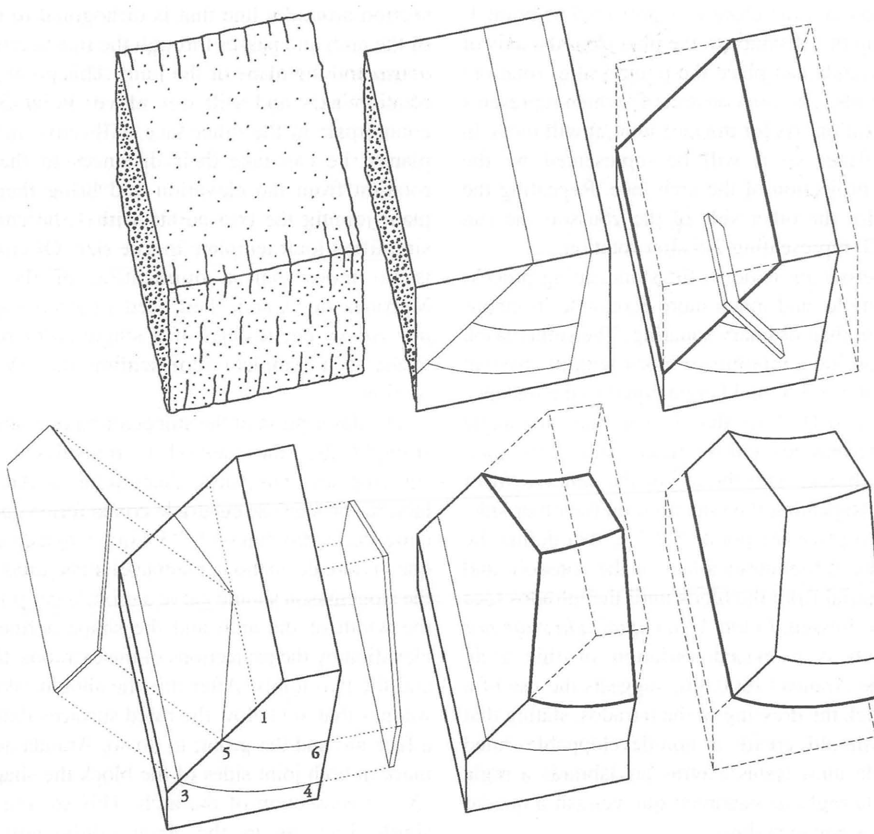

Figure 2
Dressing of a voussoir of the *Arco capialzado por robos*

projection of a *cercha*, that is, a one-sided template, to carve the intrados of the voussoir.

As always, the stonemason is to make a full scale tracing, in plan and elevation, this time of a skew arch. After that, the mason should trace true size templates of the joints and the intrados of each voussoir. As we have remarked before, Aranda uses here an antecedent of the *rabattements* of Descriptive Geometry, to avoid the disadvantages of Vandelvira's triangulations. The construction relies on a property of rotation: when rotating around an axis, a point will move on a perpendicular plane to the axis of rotation. Thus, when constructing an intrados template, Aranda makes it rotate around the lower intrados joint, 4 8. The points 4 and 8, that are in the rotation axis, will not move, but the edge of the upper intrados joint, 5, will move on an orthogonal plane to the lower intrados joint, and its horizontal projection will be on a perpendicular line to the axis of rotation. Besides, we can take the distance between points 4 and 5 from their vertical projections 3 and 2, since both are in a frontal plane, that of the face of the arch. That allows the stonemason to trace an arc with centre in 4 and radius equal to the distance between 2 and 3; the intersection of this arc with the perpendicular line will be point 5 in the template. The mason can construct point 7 in a similar manner, or take into account that the line 5 7 is parallel to the axis of rotation and will be represented parallel to 4 8 in the template; therefore, the quadrilateral 4 5 7 8 will give the intrados template.

Aranda advises to use the same method for the

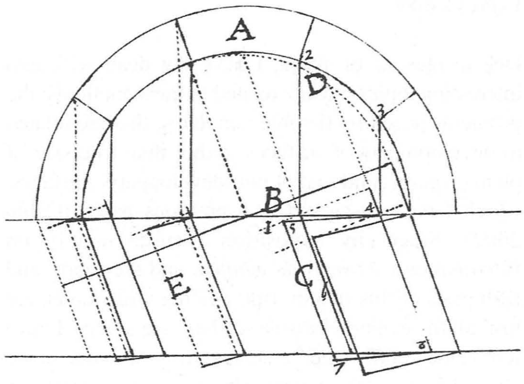

Figure 3
Ginés Marínez de Aranda. *Cerramientos y trazas de montea,* pl. 16. *Arco en viaje por cara y por plantas*

construction of the joint templates. Additionally, the stonemason can make face templates, taking them directly from the elevation of the arch, since Aranda says that «hánse de labrar las piezas por las testas con la forma que tuviere el arco semicírculo A»; that is, the voussoir should be cut by the faces with the form of the semicircular arch A. In this way, when the mason starts to dress the voussoirs, he has an intrados template, two joint templates and, if necessary, two face templates. This seems more than enough to employ the procedure suggested by De L'Orme, enclosing the voussoir in the five templates and cutting it gradually until all templates match.

However, Aranda instructs the stonemason to trace the cross-section of the arch, by a vertical plane perpendicular to the arch axis. The procedure resembles closely the *changement de plans* of Descriptive Geometry. First, the mason should trace the horizontal projection of the new vertical projection plane, in the manner of a folding line; second, he should trace reference lines, perpendicular to this folding line, passing by relevant points in the cross-section; third, he should transfer to this reference lines the heights of the relevant points, taken from the elevation. The cross-section is different from the face arc, due to the bias of the arch; in this case, since the face arc is semicircular, the cross-section is a raised half-ellipse or *arco encogido.* Once this is done, the mason can follow Aranda's advice: «por las caras de los bolsores se han de labrar de cuadrado con la forma que tuviere el arco encogido tirado en blanco D»; that is, the intrados of the voussoirs should be dressed squarely with the shape of the raised ellipse. That can be done with a *cercha* or one sided-template that is carried along the intrados, keeping it parallel to the plane of the cross-section; that is, orthogonal to the intrados joints. Doing so, the intrados surface, an elliptical cylinder, is generated as the projection, in the strict sense of the word, of the *arco encogido.*

Oblique projection and true size

Aranda (1600, 47–48) includes in his manuscript a skew variation of the *Arco por arista,* called *Arco por arista en la cara en viaje.* He starts tracing a doorway as a semicircular arch; but since the plane of the doorway is oblique to the axis of the arch, the extrados surface will be an elliptical cylinder. As in

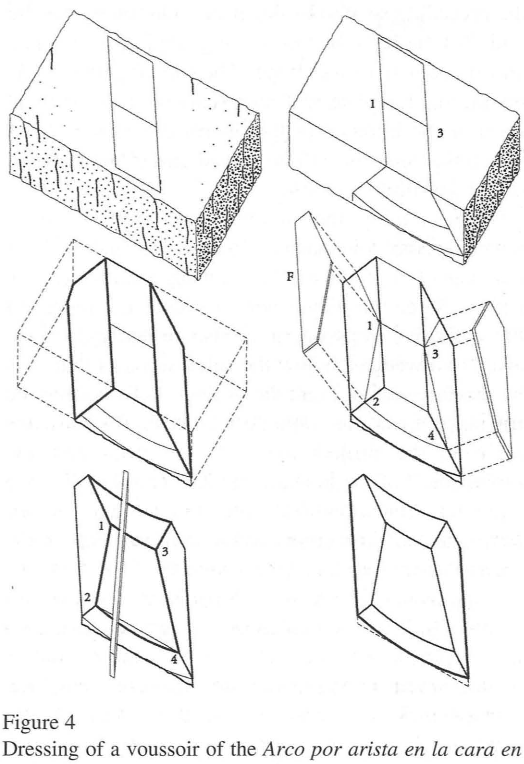
Figure 4
Dressing of a voussoir of the *Arco por arista en la cara en viaje*

the *Arco por arista en la cara*, the intrados is formed by two ruled surfaces that pass by a face arc and a groin. The elevation is not an orthogonal projection, but an unusual oblique projection. The picture plane is parallel to the arch faces, but the projecting lines are parallel to the axis of the arch; hence, they are horizontal, but biased in relation to the projection plane. In this way, both face arcs, and even the springing of the groin or *arista*, are superimposed in this strange elevation.

Aranda starts the stonecutting process by carving a block defined by points 1 and 3 and the extrados joints. However, this gives only the face side of the voussoir. He cannot carve the voussoir *de cuadrado* as he could in the previous examples, for the joints are not orthogonal to the face planes. Therefore, to pass the face template from one face to the other, as Aranda did in his theoretical explanation of the *robos* method, he needs the *planta por cara* or intrados template of the block. According to the manuscript, the stonemason should construct the intrados template by means of the procedure we have seen in the preceding section of this paper. On top of this, he will also mark the line *arista* or groin in the template and transfer it to the stone. The *planta por cara* is then a real template that represents the true size and form of the intrados of the intermediate block from which the voussoir will be carved, not of the intrados of the definitive voussoir.

When tracing the *plantas por lecho*, or joint templates, the stonemason is to construct first the joint template of the enclosing block, using *rabattements* as before. Since the *planta por cara* does not represent the definitive voussoir, it is also necessary to take away two wedges, below the ruled surfaces that pass by the face arcs 4 2 and the groin 1 3. To do that, he can take from the oblique elevation the distance between the projection 1 of the groin and the projection 2 of the face arc; transferring this distance to the representation of the face joint in the *planta por lecho*, he can get a corner of the joint template of the shaped stone. Aranda's text is short but unambiguous: «en las plantas por lechos formarás segunda vez las plantas por lechos para plantarlas al justo como parece en las plantas por lecho **F** y les robarás las piezas conforme se hizo en el arco por arista en la cara». That is, the mason is to construct a second V-shaped joint template to cut the voussoir to its definitive form, robbing it as in the *Arco por arista en la cara.*

Conclusion

Due to reasons of space, I have not dealt with two interesting topics closely related to these methods: the problems posed by flexible templates, that are related to developments of surfaces, rather than true size of planar figures; and that of non-developpable surfaces, which I have discussed in a previous paper (Calvo 2002). Since my exposition focuses mainly on *Cerramientos y trazas de montea*, and the fourth and fifth parts of this manuscript, dealing with vaults, are lost, all the examples discussed here are arches. I have left aside vaults and a stonecutting instrument so characteristic as the *baivel*, which was used mainly on vaults and *trompes*.

Nevertheless, I hope that these examples are sufficient to illustrate some basic points. Spanish stereotomic manuscripts of the 16th century, as French treatises of the period and the subsequent centuries, prefigure many central notions of Descriptive Geometry. They not only employ orthographic projection with remarkable ease, but also templates in true size, constructed by procedures that are conceptually similar to modern day *rabattements*, rotations and auxiliary views or *changement de plans*.

If the discussion of squaring and templates in the treatises of Derand (1643, 18–21) or Frézier (1737, 2:11–15) is akin to a Descriptive Geometry textbook, their use in Vandelvira or Aranda reminds more of a workbook or a collection of exercises. Martínez de Aranda only deals with general principles on one occasion (1600, 113–114) and even then only by means of an example. Of course, Aranda does not use the modern word *proyección*, and designates the horizontal projection as *plomo*, that is, «plumb line». When he needs a word for the vertical projection, he has to resort to a term so far apart from the masons' jargon as *imaginación* (Martínez de Aranda 1600, 85).

The lack of conceptual discussions contrasts with the ease that Vandelvira, and especially Aranda, show when combining the most suitable tools to solve a particular problem, whether orthogonal or oblique projections, rotations, *rabattements*, or triangulation. This strongly suggests that squaring and templates, *équarrissement* and *panneaux*, *robos* and *plantas*, are mainly didactic categories. Though they were present in the jargon of ordinary masons, as the titles of many

trazas make clear, they were not incompatible, and in everyday's work, masons probably combined them freely, as Rabasa has remarked about modern stonemasons (2000, 158).

As stated at the beginning of this paper, stereotomy has been held to be a forerunner of Monge's double-projection system. Furthermore, the examples discussed here show that the role of stonecutting in the development of descriptive geometry can be approached from a wider perspective. A secular practice, dating at least from the 16th century, employed in their everyday tasks by Spanish and French stonemasons, furnished Monge and his followers not only with the method of double orthogonal projection, but with a whole **integrated** system of geometrical problem-solving tools, including rotation, *rabattements* and even *changements de plan.*

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Evolution in the construction of facing brick façades in Valladolid with the introduction of pressed brick, from the last quarter of the 19th century to the first quarter of the 20th century

M. S. Camino Olea

At the end of the 19th century, facade walls were erected with the heading bond, so-called «a la española». The thickness of these walls exceeded one bat.[1] Because of this, they were carried out by two bricklayers working at the same time, one of them on the exposed surface of the wall and the other on the non-exposed surface. Both of them laid bricks alternating whole bricks with bricks cut in half widthways. The holes left between these two faces were regular, and they were filled with one or more headers, preventing continuous vertical joints from occurring in adjacent courses in an attempt to interlock the brickwork.

This type of wall was effected with the same type of brick until pressed brick appeared. Pressed brick was used to erect the outer face, whereas handmade or common brick continued being used for the inner face. Pressed bricks were laid with butt joints of about 3mm thick, and handmade or common bricks were laid with ordinary joints of about 8mm. In order to implement these walls on level courses, bricks of different thickness were used, pressed bricks were

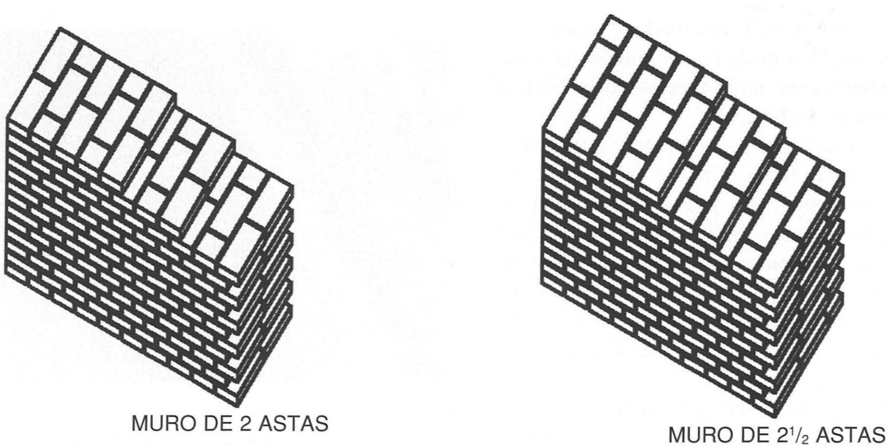

Figure 1
Pictures of a 2 bat wall and a 2 1/2 bat wall with the bond described above

5mm thicker than handmade or common bricks, and they generally had bed of similar size.

To define the building system used in these facades, the following expression was used: «**fábrica de ladrillo prensado trasdosado de recocho**» [pressed brick brickwork with a non-exposed hard burned brick facing].

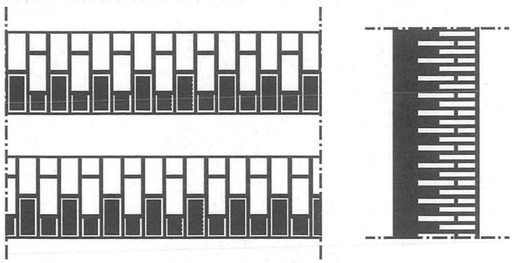

Figure 2
2bat wall effected with pressed bricks and handmade bricks. Pressed bricks in horizontal sections have a diamond pattern on the bedding surface and those in the cross-section have a striped pattern. The figure shows the two different bond courses

Facades which had been built with the same type of brick started being effected with two different types of bricks. These bricks presented several differences relating implementation, function and finish, among which the following can be highlighted:

- the horizontal wood frame made of joists rested only on the handmade or common brick inner face, whereas the outer face stood in front of the frame as a whole.
- in the facade holes, carpentry was placed between both faces. On the outer face, the recess was built with pressed brick of 1 bat width, whereas on the inner face it was with handmade or common brick. On each face, the holes were closed with different arches in the upper part: jack or segmental arches with rowlocks on the outer face, and flat arches on the inner face.
- on the outer face, the pressed brick was left exposed, and was used to devise the facade ornamental features. The inner face was cladded.

Figure 3
Pressed brick from the Silió Pottery in Valladolid found in a demolition site. Pressed bricks contained recessed panels on the surface called frogs, and the mortar which joined the bricks was placed into the frogs the bricks presented on the bedding surface. This made it possible to implement the brickwork with butt joints

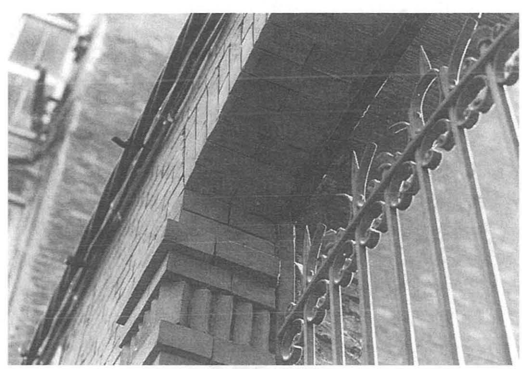

Figure 4
Photograph of a facade hole seen from the outside. The outer face (to the left and up to the gate) is pressed brick, and the inner face is common brick

The first buildings that were put up following this construction system at the end of the 19th century presented quite simple ornamental features with bricks. These were mainly restricted to band cornices at the strike with floor surfaces, the fence-in of holes, and eaves.

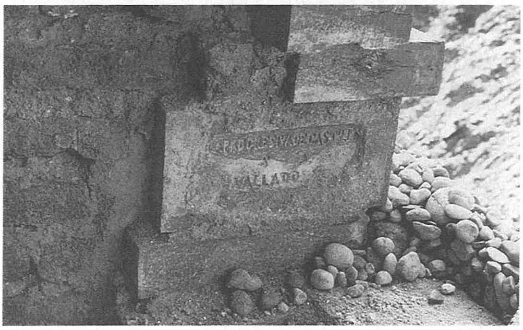

Figure 5
Photograph of a section of a building under renovation at a cornice level. The outer face (right) has pressed bricks, and the inner face handmade bricks

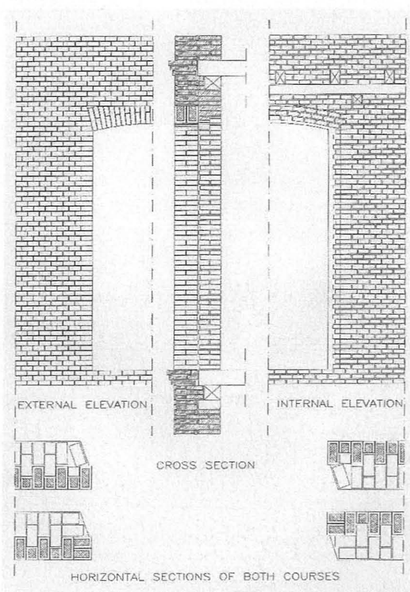

Figure 6
Descriptive drawing of the construction of these types of facades, representing a hole, and the strike of the hole with the horizontal frame (internal and external elevations, cross-section and both bond courses)

Figure 7
1883 building with ornamental features devised with two-colour bricks placed around the holes and at the strikes with the frame

Ornamental features are gradually filling up the facade external facing wall. More and more patterns are being introduced, some of which do not observe the rules for brickwork interlocking. Examples of these elements include the following:

— Head joints between bricks which are effected in the same vertical and in several courses.
— Very small cut brick pieces.
— Courses implemented with cut bricks which are sloping with respect to the horizontal.
— Vertical rowlocks with cut bricks.

There is every indication that the outer face starts being effected as a cladding face where the ornamental features are devised, whereas the inner face is the resistant one.

Another change also occurs in face thickness, since carpentry starts being placed at half bat from the

facade outer face, leaving a facing brick recess of this same width.

Figure 8
1909 building with the ornamental features devised on the piers too, and with a variety which is far superior to the one in the previous figure

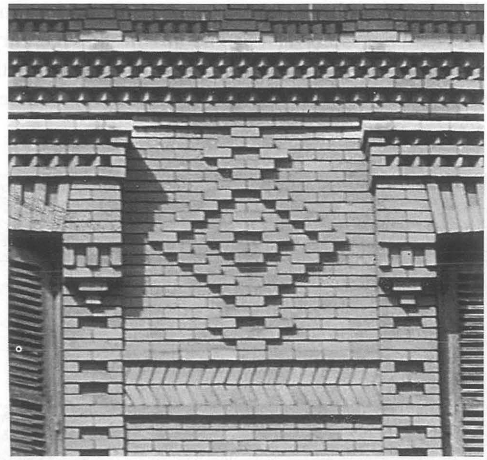

Figure 9
Detail of the ornamental features of a pier from the facade in the previous figure

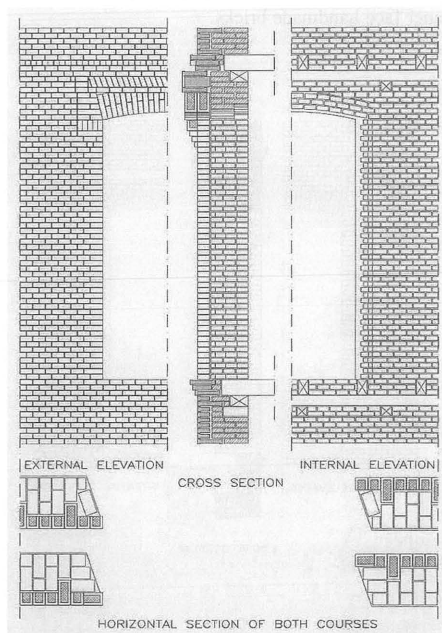

Figure 10
Bond with the outer face in half bat thick pressed brick interlocked with brick ties to the common brick on the inner face. The outer face continues being effected with header bonds but with half bricks. This implementation system is used nowadays when half bat thick faces are effected with header bonds

The building system kept on developing in the 20th century. The two faces do not have the same function in the wall anymore. One of them is the resistant one, whereas the other one is a wall cladding which interlocks with the previous face. They do not have either the same finish or the same thickness, since the outer face is half bat thick and the inner face is thicker.

This new implementation system is called «**fachada de ladrillo ordinario refrentada de prensado**» [ordinary brick facade with pressed brick on the outer face].

This building system can be considered as a precedent of the current facing brick facades, which

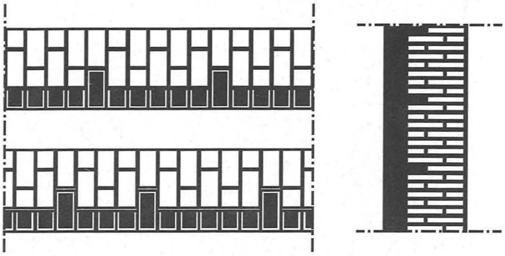

Figure 11
2 bat wall effected with the building system described above

are carried out with the so-called traditional system, and which consist of two faces: the outer face with half bat thick facing brick, and the inner face with common brick.

In summary, it can be said that at the beginning the use of pressed brick only affected the outer face of facades, since it was used in the construction of facades as a substitute for bricks placed on the outer surface. Nevertheless, and due to the possibilities this type of brick presents to devise more and more complex ornamental features, it ends up modifying the implementation of facades altogether. As a result, facades are built with a common brick resistant face and a pressed brick cladding face.

NOTES

1. 1 Spanish «asta».

The study of bricks and brickwork in England since Nathaniel Lloyd

James W. P. Campbell

The first person to write a serious history of English brickwork was born in 1867 into a comfortable middle-class family in Manchester. Nothing in Nathaniel Lloyd's early career, which included a spell managing advertising and printing in a tea company, suggested that he had any interest in either architecture or bricks. In 1893 he went into business for himself, running his own colour printing firm. This proved extraordinarily successful and by 1909, at the age of forty-two, he had made so much money that he was able to comfortably retire to exercise his twin passions: shooting and playing golf. That might have been the end of the matter had he not brought an elderly house called Great Dixter in 1910. The house was in a very poor state of repair and too small for Lloyd's purposes so he set out to find an architect to help him restore and extend it. He chose Edwin Lutygens (1869–1944) and thus began a successful collaboration which not only led to the completion of the house itself, but saw the middle-aged Lloyd, encouraged by his friend and mentor, starting a whole new career as an architect and architectural historian, interests that were to dominate the rest of his life. He went on to design a number of buildings and publish five books on aspects of architecture and gardening of which the two most important were *A History of English Brickwork* (1925)[1] and *A History of the English House* (1931). Lloyd died in 1933. His wife continued to live in the house he and Lutygens had built until her death in 1972 and it is now open to the public.

A History of English Brickwork was an extraordinary book being both scholarly and profusely-illustrated, a rare combination. Lloyd brought together for the first time, and quoted at length, the most important writings on brick from the seventeenth century onwards. He included detailed tables of brick measurements, exploding the myth once and for all that bricks could be dated from size alone. Moreover to illustrate the book he took dozens of black and white photographs of buildings throughout England (some of which are now the only evidence for the appearance of structures which have since been demolished) and included careful measured drawings of key examples. Together these form three-quarters of the book.

The book remains one of the most authoritative and useful works on the subject today. Indeed Lloyd was in so many ways a pioneer in the study of brickwork, that many still assume he has had the last word on the subject. Even today *A History of English Brickwork* is often the only work on bricks to appear on architect's bookshelves and the one they are most likely to cite in discussion. Yet a great deal of research into the history of brickwork in England has been done in the past seventy-seven years since Lloyd's book appeared in 1925. It is this research that forms the subject of the current paper, the aim of which is twofold: firstly, by reviewing the literature available, to show how scholarship has developed since Lloyd as a useful guide for those looking for more information on the subject; and second, and more

importantly, to indicate those areas which seem to have been ignored.

Before commencing on such an enterprise it is worth noting that the scope of this article is limited to the history of brickwork. Modern textbooks on the subject of bricklaying or brickmaking are excluded except where they might be of interest to the historian looking at periods before Lloyd. Now that we are into the twenty-first century, study of the twentieth century becomes ever more important and such textbooks will no doubt become the historical sources of tomorrow, but this is a paper on historiography rather than a survey of historical sources. Terracotta and tiles are excluded as not being strictly brick. Geographically I have confined myself to research done on English architecture chiefly because that was the subject of Lloyd's book, although I have extended the area to cover Scotland and Wales where this seemed appropriate. British researchers have written on brickwork beyond their shores, most notably in reports on archaeological projects on Classical and ancient civilisations in Europe and the near and Far East, but this material has not been systematically collated and remains fragmentary.[2]

Books and General Works on the History of Building Construction

Although there had already been a number of books seeking to set out the methods of building construction in various periods, the history of building construction before Lloyd was still a minor area of interest compared to the extensive studies carried out into architectural style. The field of construction history was noticeably better developed in France, for instance, than it was in England. This was one of the reasons Lloyd's work was exceptional. Construction history was to remain a field of minor interest in Britain for the first seventy-five years of the twentieth century. The expertise existed, but it rarely found its way into print. Nevertheless a few isolated general studies on the history of building construction did appear during this period and these normally included chapters on both brickwork and mortars. The introductory nature of these studies naturally meant that the amount of new research carried out into brickwork in particular was limited and most relied on secondary sources (often for brick by citing Lloyd) but they did play an important part in setting brick in the context of the building industry as a whole.

Briggs (1925) and Davey (1961) were two of the broadest studies covering the whole world. Davey focused on building materials and contains an interesting study of mortars while Briggs, whose work came out in the same years as Lloyd's book, followed a craft based approach. Both are now out of date. Alec Clifton Taylor (1972) focused on entirely on English building materials. It remains a valuable introduction to the subject and a model of erudition, although it is great pity that, for a book on the colour and richness of materials, all the illustrations were printed in black and white. Clifton Taylor went on to produce a number of titles on individual materials including one with Ron Brunskill that focused entirely on brick discussed below.

Very useful studies of individual periods of English construction history have appeared which do an excellent job of summarising previous scholarship. For the Medieval period Salzman (1952) remains an invaluable survey of the terms used and costs involved in Medieval building work compiled from an extensive search of building accounts. It remains the most important survey of the period in this respect.

Malcolm Airs (1995) provided a similar analysis of the state of the building world in the Tudor period. No comparative survey exists for the seventeenth century. The eighteenth century has been well researched starting with Dan Cruikshank and Peter Wyld's beautifully illustrated *London: the Art of Georgian Building* (1975) and more recently James Ayres *Building the Georgian City* (1998). Cruikshank and Wyld (1975) provide invaluable drawings and photographs of many buildings of the period which have since been lost, while Ayres (1998), as well as providing an excellent overview, reprints in colour many contemporary illustrations from the period and later providing a useful source for visual references.

The complexity of the nineteenth century seems to have discouraged English scholars from writing general works on building construction for this period although perhaps not surprisingly it has attracted more interest in America. What has been written tends to be in volumes devoted to particular materials, those on brick being noted below. A very useful survey of building construction in the twentieth century is found in Yeomans (1997).

General Books on Bricks and Brickwork

There have been disappointingly few books devoted entirely to the history of English brickworks since Lloyd. Of the general studies that have appeared the most important is undoubtedly Professor Ron Brunskill's *Brick Building in Britain* (1997) which was designed as a revision of and replacement for an earlier work, *English Brickwork* (1977), which he had co-authored with Alec Clifton-Taylor. *Brick Building* contains chapters on the history of brickmaking, bricklaying, an excellent glossary and sections on brickwork of different periods. It is a model of research and cautious in its approach. Three short appendices, one on brick tax, one on cavity walls and one on brick in Scotland are model essays on their subjects. It still remains the best summary of scholarship on the subject and essential reading for all those looking for an introduction and contains an excellent bibliography.

Woodforde's *Bricks to Build a House* (1976) is an enjoyable introduction to the subject, but has a rather cavalier attitude to historical facts. It was designed with the interested layman in mind rather than the scholar or conservation professional. Where it does have value is in its illustrations, particularly its collection of useful nineteenth century engravings. More cautious in its approach is Hammond (1981) which provides an excellent introduction to all aspects of brickmaking, but at only thirty-two pages long is too slim to go into any depth.

More recently Andrew Plumridge and Wim Meulenkamp's *Brickwork* (1993) has provided an overview of all aspects of brickwork across the globe illustrated by lavish colour photographs and including an excellent forty page section on construction and materials and a short history. Plumridge is English while Meulenkamp is Dutch. The book is thus an unusual example of international collaboration, which is surely something that should be encouraged. Nevertheless some reviews of the work have been less than complimentary about its scholarship (T. P. Smith 1994b).

Also worth mentioning are Gerard Lynch's *Brickwork* (1994), John Warren's *Conservation of Brick* (1999) and volume two of John and Nicola Ashursts' *Practical Building Conservation* (1988), all books that are aimed at the conservation architect. These provide information on the technical side of brickwork restoration, together with short histories of brickwork.

Lastly, two American books are worthy of note because they have direct bearing on English brickwork. The first is Karl Gurcke's *Bricks and Brickmaking: a Handbook of Historical Archaeology* (1987). Its primary focus is on the development of brickwork in America, but it does provide insights into the mechanisation of brickwork in England and includes an invaluable guide on distinguishing bricks made using different factory methods. The second is Joseph Arnold Foster's *Contributions to the Study of Brickmaking in America* printed in six volumes from 1962 to 1971 which, despite its title, devotes the first four volumes to reprinting exclusively English sources from 1600–1850. Unfortunately the book was privately printed in runs of two hundred copies or less, making it almost as hard to get hold of as most of the sources it is reprinting.

Dictionaries and Enclyclopaedias

A survey of every entry in every dictionary under brick or brickmaking would be unlikely to yield anything of interest, but one entry that is worth mentioning is the excellent section in the *Grove Dictionary of Art* (Turner, 1996) under «brickwork» which includes contributions on moulding, firing, bonding, diapering, and sections on the history of brick in various parts of the world.

As far as I know, no-one has thought to produce a dictionary exclusively devoted to brickwork, which would undoubtedly be useful, although probably not a bestseller. The closest I have come across is Searle's three volume *An Encyclopaedia of the Ceramic Industries* published in 1929, but this contains no information on the history of the brick.

Detailed Studies of Bricks and Brickwork since Lloyd

There is a growing call among architects, engineers and others working in the conservation world for better books on construction history. Professionals in these fields have little time to visit libraries during the working day and frequently find scattered papers in academic journals too difficult to find to make it

worth their while. Lloyd's book was aimed at just this audience. Yet ironically it is in the area of academic papers that Lloyd's work has done the most to encourage publication while the popularity of his book has acted as an obstacle to the publication of further books on the subject. Scholarship has moved on and this may be about to change. A group of enthusiasts originally centred around Laurance S. Harley (1901–1983) have acted as a catalyst for encouraging research since 1972 and are leading research into brickwork.[3]

The British Brick Society started with just four members members: Laurance.S. Harley (who was an engineer by training but involved in archaeology), Geoffrey Hines (a humanities lecturer in Adult Education), Ron J.Firman (a lecturer in geology at Nottingham University) and a young archaeologist from St. John's College, Cambridge called Terence P. Smith. It was partly conceived as a specialist study group of the British Archaeological Association, but from the outset it welcomed members from any background. Its constitution set out a number of aims including: the study of bricks and brickwork from Roman times to the present day; some investigation of the precursors of the baked brick; the study of continental bricks and brickwork; the encouragement of a multi-disciplinary approach including archaeological, architectural and scientific studies of the material and its uses; the investigation of geological, physical and chemical ways of dating bricks; the preservation and conservation of brickwork; and the establishment of a system of archives and records on the subject which would be made accessible to the public.

From the outset the group produced a regular newsheet simply termed *Information.* Both the newsheet (now a full-blown journal). It is currently produced three to four times a year and sent to subscribing members and major libraries, has since its conception been one of the most important outlets for research on the subject, intermixed with more general queries and observations. It also contains regular reviews of relevant literature. Many of the Society's members publish in other journals devoted to particular subjects and since 1973 publications on brickwork have increased noticeably to the extent that it is possible here only to summarise the extent of present knowledge and to review key articles on particular subjects which in turn provide further bibliographies which the interested scholar can follow up on particular topics. The rest of the paper will be devoted to tracing how these and other papers have advanced scholarship in various areas since Lloyd, starting with brickmaking, then social and economic studies, geological and scientific studies and finally the study of bricklaying and architectural brickwork. The aim is to provide a clear overview of the subject as it now stands in order to highlight those areas that could benefit from further research.

BRICKMAKING THROUGH THE AGES

The baking of bricks to use in building construction has a very long history but the story of brickmaking in Britain begins with the Romans. Lloyd pointed this out, but as the full title of his book suggests, his treatment of Roman bricks was brief and even Clifton Taylor writing in 1972 could add little of substance. Since that time, however, the study of Roman bricks has advanced hugely. The Romans stamped some of their bricks with distinguishing marks which can be used for dating. Brick stamps have thus proved invaluable in archaeology and there is a large literature on the subject. A series of excellent articles summarising scholarship (including excavations of major kiln sites) are collected in McWhirr (1979). Roman brick production in Britain is reviewed in Darvill and McWhirr (1984) which also provides an extensive bibliography. A further summary of all this material and a longer bibliography can be found in Gerald Brodribb's *Roman Brick and Tile* (1987) which despite its title is entirely devoted to bricks produced in Britain under Roman occupation. The Roman legions appear to have been responsible for running brickyards in Britain, so that when they left, the craft of brickmaking went with them.

The Anglo-Saxons and Normans were content to re-use scavenged Roman bricks on various buildings (for instance St Botolph's Priory, Colchester and St Albans Abbey). A distribution map of such instances is given in Smith (1996) and a discussion in Smith (2001). Brickmaking was not revived until the end of the twelfth century when new bricks appear to have been used in Polstead Church in Suffolk and Little Coggeshall Abbey, Essex a few miles away (Lloyd had mentioned Coggeshall in his book). A nearby brick kiln is said to have been excavated in the

nineteenth century and destroyed. The Coggeshall brickwork is reviewed in J. S. Gardner (1955). Polstead is noted in Wight (1972) and reviewed in Kennett (1990).

Medieval brickmaking has been extensively studied by T. P. Smith in his *The Medieval Brickmaking Industry in England 1400–1450* (1985b) which has a good bibliography of both English and Dutch secondary literature. Salzman (1956) provided an introduction to terms used in the period while the accounts of the medieval kilns in Hull are analysed in Brooks (1939). Excellent summaries of the medieval brickmaking industry are also found in Drury (1981) and Moore (1991).

Although the brick earth was dug and mixed by hand, the exact method of moulding is not recorded and has been widely discussed.[4] Once moulded the bricks were set out to dry before firing. Both the methods of stacking and moulding leave marks on the bricks which were discussed in Hammond (1986) and Firman (1986).

Medieval pottery and tile kilns have survived,[5] but no specific brick kilns from the period have been excavated to date. Clamps were also used for firing bricks in the Middle ages but by their nature they leave little trace behind them.

Brickmaking in the late fifteenth and sixteenth centuries is surveyed by Howard (1991). By the seventeenth century, brick was widespread and used for smaller houses and churches leading to an increase in production. The first printed references to bricklaying and brickmaking (some of which were printed by Lloyd), appear in England in this period and are listed in Campbell & Saint (2002). Since Lloyd's time three important manuscript sources have been published, namely the accounts associated with Christopher Wren published in the twenty volumes of the Wren Society (1924–43) and the notebooks of Roger Pratt (Gunther 1928) and Roger North (Colvin and Newman 1981).

The Great Fire of London in 1666 and the numerous fires in other towns around the country prompted increased legislation in favour of brick over more-flammable timber-framing. Some of the London regulations were reprinted in Lloyd, but T. P. Smith has since shown that they were rarely followed (Smith 2000a). Brickmaking after the Great Fire is discussed in Cox (1989), (1997) and Yeomans (1987). Accounts for brick supplies for Hampton Court are discussed in Musty (1991) and for the West Midlands in Whitehead (1981). A seventeenth-century contract for brickmaking is reprinted in Kelsall (1983).

Kilns have been excavated from the seventeenth century and later (see Drury 1975) as have clamps (see Wade (1980) and the photographs of a clamp excavation in London in Ponsford and Jackson (1997, 316–317) and the notes in Ponsford and Jackson (1995, 179).)

One of the innovations in this period was the use of ash added to clay to make London «stocks». Their manufacture is discussed in Cox (1989) and (1997). A general description of brickmaking in the Georgian period together with illustrations can found in Ayres (1998).

By the eighteenth century mapmaking had progressed to the extent that brickmaking sites are discernable. It has thus been possible to compile regional gazetteers of brickmaking sites for the period c.1700–the present, with some earlier sites being located from other sources. Those gazetteers that have already been completed are reviewed in Kennett (1999) and (2000). They often provide detailed local histories of brickmaking and tend to be done by county. The first to be compiled was Hampshire (White 1971), followed by Bedfordshire (Cox 1979), Buckinghamshire (Pike 1980), Oxfordshire (Bond, Gosling & Rhodes 1980), Suffolk (Pankhurst 1988), Somerset (Murless 1991), Sussex (Beswick 1993), and Essex (Ryan 1999). A separate study (Douglas & Oglethorpe 1993) covers all of Scotland. Local studies of individual districts in Surrey, North-East Hampshire, Acton, Burton-on-Trent and around Ascot are also listed in Kennett (1999).

From 1784 until 1850 bricks were taxed in England. The tax had a number of important effects on the industry. As the tax was imposed per brick it led to an increase in the size of bricks until this was countered by further legislation (Exwood 1981a). There have been a number of studies of the effect of the tax, most notably: Exwood (1981a) and (1981b), Shannon (1934, 188–201), Smith (1992a), Smith (1993), and Smith (1994a). A summary is included in Brunskill (1997, 192–93). The myth that the brick tax led to the introduction of so-called «mathematical tiles» (thin tiles which look like bricks) has been disproved by Smith (1979) and (1985a); and Exwood,

(1981b), (1985a) and (1987).[6] Kennett (2001) deals with why the tax was abolished.

Lloyd's interest in the development of brickmaking ended at the end of the Georgian period. Of course today the interests of many researchers begin where Lloyd left off. In the nineteenth century brickmaking began to mechanise. The shift was a slow process and bricks were still being made by hand into the twentieth century and in isolated instances they are still made by hand today. The first treatise on brickmaking to show mechanisation was written by Edward Dobson in 1850 and was reprinted in full with a useful introduction and bibliography in Celoria (1971). Because of the longevity of many techniques some early twentieth century books for the contemporary brickmaker also provide useful sources for nineteenth century practice. This is certainly the case with Alfred Searle's *Modern Brickmaking* (which was first published in 1911 and went through no fewer than four editions before 1956) and the entries found in building manuals by McKay and Mitchell noted in Brunskill (1997, 199).

Kilns underwent great changes in the nineteenth century. Details of types of kiln and clamp can be found in Celoria (1971), Hammond (1977, 1981, 1984a, 1984b, 1987 and 1988) and Searle (1911). Hamer and Leslie (1991) and Andrews (1986) provide details of two very different types of works and Ryan (2000) gives a list of references to nineteenth and twentieth century manufacturers published in the last thirty years in *Information.* Histories of individual companies have also been published, most notably those by Hillier (1981), Christian (1990) and Cassell (1990).

A great number of Patent bricks were invented in the nineteenth century. Hammond (2000) looks at Cartwright's «Interlocking Bricks». Smith (2002) provides a survey of the use of «Hiort Patent bricks» and Storey (1970) looks at «Hitch Patent Bricks». Many others have been mentioned in passing in various articles in *BBS Information* and listed in the index (Ryan 2000)

Bricklayers and brickmakers: socio-economic studies

Despite the fact that much information in the form of building accounts survives from the Middle Ages onwards to allow a detailed picture of the economic status of the building craftsmen, there has been comparatively little analysis of this data. Medieval accounts are reviewed in Salzman (1952), Smith (1985b) and Moore (1991). The situation in post-medieval northern Britain is discussed in Woodward (1995) and in London for a similar period in Summerson (1945) and McKellar (1997). The eighteenth-century situation is discussed briefly in Shannon (1934), Smith (1984) and Ayres (1998). A treatment of the changes in the economic structure of the industry in the period can be found in Clarke (1992) and for the nineteenth and twentieth centuries in Powell (1980). The publication of specific building accounts are mentioned under bricklaying below.

Guilds undoubtedly played an important part in regulating early medieval urban brickmaking and bricklaying. Guild records survive in many towns in Britain but virtually nothing has been written on the subject. A list of the apprentices taken from the books of the London Tylers' and Bricklayers' Company is published in Webb (1996) and is mainly intended for genealogists. Bell (1938) provides a short history of the London Company. The rules and regulations of the guild in Hull had been reprinted before Lloyd in Lambert (1891, 272–282).

Geology and Analysis of Bricks

L. S Harley, founder of the British Brick Society, was keen to see the development of better methods recording and analysis of bricks discovered during archaeological investigations. He set out his own detailed method for recording bricks in *The Journal of the British Archaeological Association* in 1974 and this remains the most detailed typology yet set out, although it is rarely followed in practice. A rare example of the type of analysis Harley envisaged can be found in Ryan and Andrews (1993) which is reprinted in Warren (1999, 59–68).

One of Harley's recommendations was that the type of clay should be recorded. There has been a great deal published on the selection of clays for the modern brickmaking industry but comparatively little has been written on the analysis of clay for determining the origin of bricks. The foremost advocates of this practice are husband and wife team Ron and Pat Firman who first called for this approach

in an article in *Mercian Geologist* in 1967. Subsequent articles on the same subject include Smalley (1987), Firman and Firman (1989), Firman (1994), and Firman (1998). Sadly geological analysis of bricks has not been generally taken up although one fine example of an investigation of this type is found in Pavia et al (2000).

BRICKWORK

Nathaniel Lloyd's *English Brickwork* had included detailed drawings and photographs of surviving buildings from the period of the Middle Ages up to circa 1800. Lloyd undoubtedly saw his book as being about architecture and for architects and the same attitude was taken by many of the books which we have already discussed. In such a scenario brickmaking is important in relation to how bricks are used, yet in the literature on brickmaking the focus is on the brick as a product of an industrial process. Many of the British Brick Society's members are or have been involved in brickmaking or are active collectors of bricks as artefacts and it is perhaps inevitable that their interests lie primarily in the history of production rather than use. Architectural historians meanwhile tend to come from an art history background and are both less interested in building technology and more prone to focus on architectural style. Nevertheless studies of the use of bricks in buildings do exist and are worth briefly reviewing here.

On the Middle Ages Jane Wight (1972) provided a useful survey of the major buildings constructed before 1550, updated for Eastern England by Harley (1975/76), and Kennett (1988). T. P. Smith's analysis of the Rye House, Hertfordshire (Smith, 1975) provides an excellent example of how recording should be carried out. The accounts for Caister Castle were published in Barnes and Simpson (1952), and those for Tatershal castle, in Simpson (1960). A portfolio of full size drawings of brick details was produced by Small and Woodbridge (1931).

Regular bonding in English brickwork seems to have been a relatively late development. An excellent analysis of the types of bonding employed and their distribution can be found in Brian (1972) and (1980). On the use of Flemish bond (an early 17th century innovation) see Kennett (1984). Diaper patterns were popular in the Tudor period. Studies of diapering are found in Smith (1985b) and (1992b). The subject of brick infill in timber buildings (nogging) which was widespread in this period is discussed in McCann (1987).

The great innovations of the seventeenth century were the shaped gable, flemish bond, and rubbed and gauged brickwork. The foreign origin of shaped gables was discussed in Hitchcock (1978) and their importation to England in Kuyper (1980). For the general Dutch influence on English brickwork see Percival (1989). For a fine example of analysis of a surviving seventeenth-century building can be found in Smith (2000b) and an analysis of bricklayers' contracts from the period in Campbell (2002).

An explanation of the methods used in gauged brickwork is found in Lynch (1990). A detailed study of this subject is overdue, although a number of excellent examples are recorded in Cruikshank and Wyld (1975) and Small and Woodbridge (1931).

From the nineteenth century onwards the involvement of architects in the design and specification of brickwork means that studies of the subject are usually included in works on the architects themselves. Studies of brickwork in isolation are rare.

One important innovation of this period was the cavity wall which was to become the standard way of building brick walls by the middle of the twentieth century. An excellent account of its development and bibliography are provided in Brunskill (1997 193–196). The best historical accounts of the general use of brickwork in Britain since 1900 have been provided in Yeomans (1997) and Kennett (2001a, 2001b, & 2002).

SUMMARY: GAPS IN THE LITERATURE

From this all too brief survey of the literature of the history of bricks and brickwork in England a number of clear gaps in the literature become immediately apparent. For the Middle Ages much has already been done on manufacturing, but remains to be done on the recording and use of brick, on the types of clay employed and on the influence of the guilds. For the later periods there are many gazetteers for counties still to be written. A proper historical study on the history of the brick kiln for not just England but across the globe is overdue. To date, there has to been

little written on the manufacturers, suppliers and types of brickmaking machinery that appeared in the nineteenth century and how they were taken up. There has been no publication of English makers marks to parallel that for America in Gurcke (1987) despite the fact that this would be invaluable for dating building fabric. The fascinating subject of Patent bricks is worthy of a book in its own and has hardly been touched upon.

Studies of brickwork are less common than those of brickmaking. There is much work still to be done on bonding patterns and their distribution and on the development of pointing and mortar used. Brickwork is rarely carefully recorded in contrast to timber framing, for instance, and there seem to be no generally recognised guidelines on how such recording should be carried out. Lastly, but by no means least, the chemical, geological and physical analysis of bricks is an area that still remains to be developed.

Today the brick industry is struggling. It has enormous capacity but it is facing a decreasing demand for its products. Some might say that this is self-imposed as the bricks the industry produces have become less appealing and the manufacturers less flexible in reacting to what architects want, but architects themselves are poorly informed of the advantages and possibilities of brickwork and there are fewer and fewer skilled bricklayers to carry the work out. The history of bricks and brickwork does not hold all the answers to these problems, but a better understanding of it is at least important in providing a framework for discussion.

Notes

1. The full title is *A History of English Brickwork. with Examples and Notes of the Architectural Use and Manipulation of Brick from Mediaeval Times to the end of the Georgian Period*. The book has been republished twice since. The first reprint abridged by Leslie Mansfield in 1934 omitted pictures and reduced the text. It is to be avoided. The second reprint by the Antique Collectors Club issued in 1983 is a complete facsimile of the original.
2. An excellent French guide to papers on brickwork in Mesopotamia has recently appeared including an extensive bibliography (Sauvage, 1998). As far as I know such studies have yet to be written for China, SE Asia, and India. The British have noticeably lagged behind in the study of European brickwork and with the exception of articles on Dutch architecture I know of little of substance in this field.
3. The history of the Society is taken from Kennett (1993).
4. Certain types of moulding lead to «sunken margins» the origin of which were discussed in a number of articles listed in and concluding with Betts (1996).
5. For an example see Drury and Pratt (1975).
6. See also Nail (1996).

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The iron staircases: Light and inventive solution of historical architecture

Tiziana Campisi
Calogero Vinci

THE IRON STAIRCASES. HANDICRAFT MANUFACTURE AND INDUSTRIAL PRODUCTION, BETWEEN THE XVIII AND XIX CENTURIES

The diffusion of elegant metallic structures like the iron staircases, the penthouses, the greenhouses, the «winter-gardens», usually considered like furniture elements rather than like real complements of architecture, it contributed, in the second half of XIX century, to make familiar the new iron architectures. In many cases in fact, the custom with the full and soft forms of stone monuments, allowed that the linear geometries of iron structures weren't appreciated, especially if these were applied to building types, for which constructive techniques and aesthetical canons were, by now, encoded from centuries of experience and constructive practice. For this reason, the iron use with structural function in the monuments and in the public buildings like the theaters was carefully dissimulated.

In the staircases, instead, the demand of that «apparent solidity», searched and recreated in an artificial way through structures that simulated, either in the volumes or in the decorations the masonry constructions, didn't showed itself.

In Sicily, at the end of the XVIII century the iron use at sight was limited to the corbels and handrails of the balconies and to small staircases. These last, also if were characterized by reduced dimensions, when presented esteem characters, like particularly elaborate banisters, it could be considered an element that dignified the building. So, the presence of these structures was a characteristic of the «*domus magna*» (according to the coeval documents we have examined), as well as the engraved stone-frames, the balconies with stone corbels, stone-slab like flooring and the big windows. In the same period very steep iron staircases were realized diffusely, according to the type with ribs and rungs inside the defensive towers or bell-towers, and agile service staircases that allowed the access to the inside rooms or to little gardens and courts set to different altitude in many buildings that was interested by repeated configuration in all the XVIII century. (Figure 1)

Is this the case of the eighteen century *Merendino-Costantino* palace in Palermo, in which a similar staircase, placed in a small inside courtyard, connected the middle floor with the superior floor. (Figure 2)

This very steep staircase, similar to these that facilitated the inspection of the coeval «urns» located over the «water-distribution towers» (called also «water-towers», or «*castelletti*»), characterizing the historical urban tissue, was so realized: the main structure was constrained to wall through flat iron triangular corbels («*righettone*» flat, «*a sguscio*» corbel); the edge-flats, constituted by a double «*righettone*» flat (having dimensions of 4,5 × 1,5 cm), were constrained to these corbels through pins and nails, that were subsequently clinched. The handrail iron vertical rods were connected to these double iron flat and were

Figure 1
Iron staircase of the Trabia castle (Trabia, PA)

Figure 2
«Water-distribution towers», located in D'Ossuna square, in Palermo

performed with the same type of iron. The steps were realized also using a flat iron of small dimension («*righetta*» flat), as well as the inclined connections between the aforesaid iron vertical rod.

Because of the lacking minings resources, the transport difficulties and the elevated costs of the raw materials in Sicily, up to the first half of XIX century, the foundries in Palermo were specialized only on handicraft production, inadequate to the melted-iron workmanship for building use.

Although in the second half of the nineteen century the local building employment of this material was still limited to sporadic applications, structures designed on French or English models or directly imported from other Countries had notable success, also thanks to foreign specialized magazines that not only divulged new techniques, but above all proposed finished products. Not casually many of these structures were directly imported from France and then assembled «on side». The first panormitan foundries, the *Oretea*, founded in the 1841, the *Gallo* (1842), the *Maggio* (1850) and the *Panzera* foundries (1870) should compete with foreign Firms, like the French *Izambert*, privileged as regards the first not so much for a superior quality of the products, as for the

Francophile predilection fed from the diffusion of magazines and catalogues that supported the France myth, and particularly of Paris like capital of the good taste. In the tables that illustrate the numerous solutions and brevets of constructive systems for the iron and cast iron staircases, often, there wasn't a specific reference to the plant, and this circumstance underlines that for any adopted form, whichever was the conformation of the staircase space, and the landings there were or not, it was possible, varying the combination and the assemblage of the parts, to maintain the same structural warping. It was the applicative case of the staircases with rack-sides, shaped in a different way in function of the flight development. In this way, for any ordinary dimensions plant used for the eighteen century residences, cutting the sheet according to the plan dimensions, it was possible to realize a staircase in brief times, also intervening in a very limited way on the existing structures. (Figure 3)

In considerations of these advantages, an obstacle to the diffusion of this staircases type often was represented by the more elevated cost as regards the wood staircases, but the reduced expenses for the maintenance surely turned to advantage of the first type.

If the reduced dimensions of the metallic frame parts, the realization rapidity, the exceptional versatility of the industrial semi-worked product and the simple execution of the connections imposed the importation models, the local diffusion of the constructive techniques connected to the iron use, often revisited by the local planners talent and by the artisans mastery, contributed to the wish of the type creation and a critical adaptation to the local demands; this condition carried a new rush for the local production of original elements.

An example was represented by the symbiotic relationship established between industry and craftsmanship for the iron coverages realization of the two big panormitan eighteen century theaters, the *Politeama Garibaldi* designed by G. Damiani Almeyda and the *Massimo Vittorio Emanuele* designed by G. B. F. Basile, in which the iron workmanship was efficiently exploited for adapt elements industrially produced to the designers demands.

This industrial product awareness, like a raw material that should suffer some ulterior elaborations,

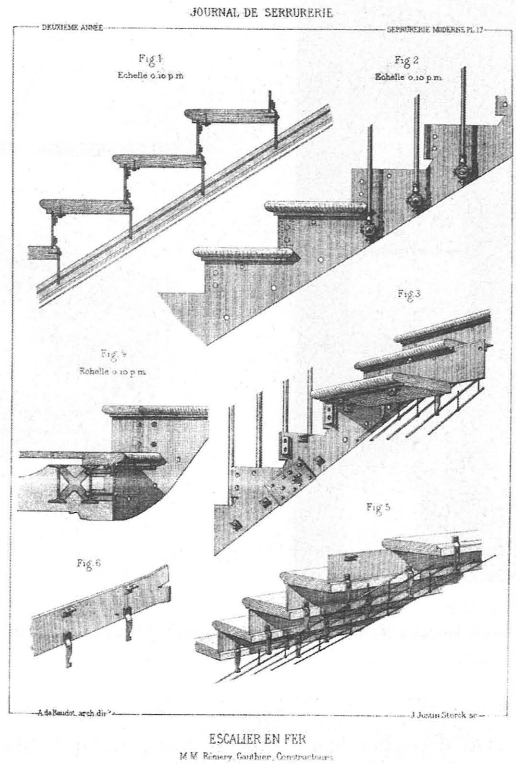

Figure 3
«Rack-sides» iron staircase. (*Journal de serrurerie* (1875), col.17, pl.17)

continued for all the XIX century, also in the small iron works of the more inspired sicilian engineers and architects. So in 1898 the architect Ernesto Basile, in the amplification project of a rural residence in the Agrigento province, he characterized his intervention by the use of industrial production elements and through these he designed an elegant winding-staircase that represented a junction point between stylistic research, craftsmanship and industry. (Figure 4)

It seems evident that, till the iron would be employed like a stone or wood substitute, the distance between material and its expressive form would had perceived in an evident way. The same architect Damiani underlined that the architectural form should imposed itself to the mechanics and not this to that.

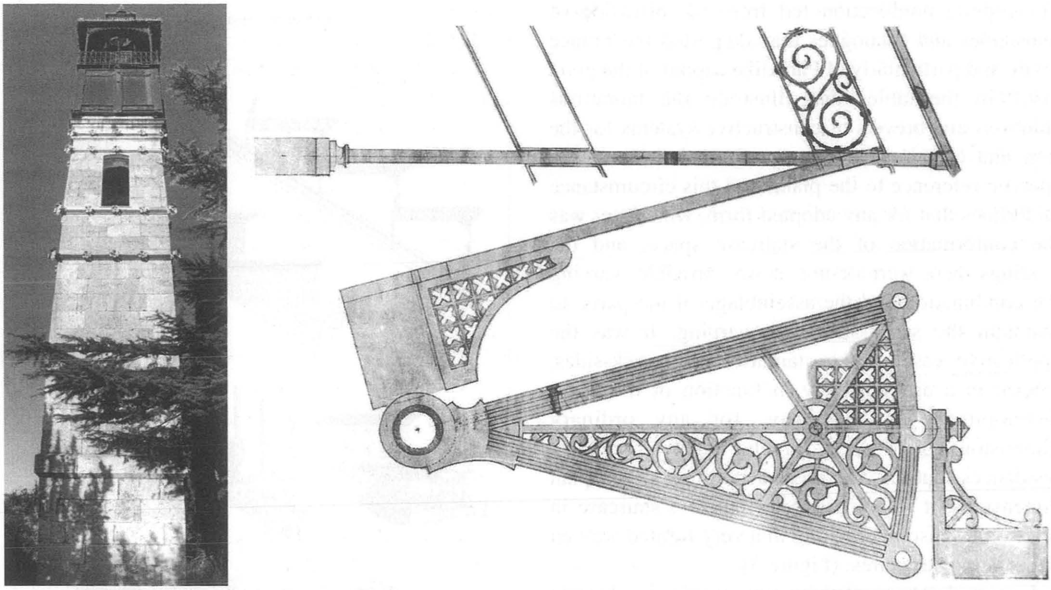

Figure 4
Clock tower and handrail of the winding staircase designed by the architect E. Basile for a rural residence in the Agrigento province.

On the other hand, it was already clear to the contemporary that in the staircases and in many other small iron and cast iron structures, the slenderness and the mouldableness, that characterized these materials, were exasperated in an artificial and redundant way, making a misuse of ramificated beaten iron ornaments and printed or fused decorations. (Figure 5)

The contemporary also reproached to the metallic structures an useless complexity as regards the structure and the assemblage systems, resulting often superabundant, revealing a certain distrust, above all for the big structures, towards the mechanical characteristics of the not forged iron.

The advantages offered by the iron and cast iron staircases were fundamentally the lightness, the

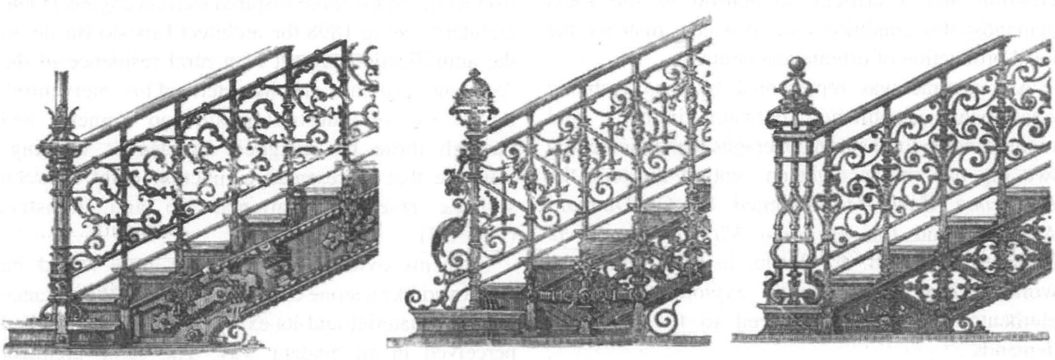

Figure 5
Examples of ornamental iron staircases. (Donghi, vol.I, p.II, pag.294)

adaptability to different planimetric fittings, the space economy, the solidity, the possibility to realize structures of a certain elegance and the incombustibility. The iron employment for the staircases construction remedy to the inconvenients deriving from the wood use, that often was adopted not perfectly seasoned; this condition made unfixed the connections, also provoking stability problems and made unsteady the staircases.

Also the incombustibility of the metallic structures was appreciated limitedly to the comparison with the wood staircases, that generally just those made with iron should replace them; in a first time, in fact, the iron was used, denying the own superior ownership and structural potentiality, like wood.

The presumed fire resistance of the iron staircases was much debated. It was evident that the notable deformations which these structures were subject, the surfaces that overheated and the possibility that the cast iron parts, of which the steps were constituted, could break because of the changes of temperature suffered at the moment of the extinction, all these conditions made these impracticable in case of fire.

Besides, the prerogative of the incombustibility imposed that the staircase cage was built with masonry and that in the cage weren't openings at the floors pianos, if not those tightly necessary to guarantee the access. The skylights prevented that the staircase becomes impracticable because of the smoke.

After some disastrous fires, that showed the iron staircases ineffectiveness to guarantee the safety of those people that occupied the building, different systems were experimented for protect the metallic structures. Respect to the hollow structural elements, like the winding-staircases nucleuses or the columns that support the landings, it seemed useful to fill the hollow with cement or in a better way with clay or dries sands. Other systems, like the circulation of air or water in the inside of the hollow columns, showed themselves complexes in the realization and little effective. For avoid the breakage of the cast iron elements not protected, it was preferred to use those who were placed straight and not horizontally, for guarantee the material homogeneity in radial direction. For the iron structures, the main problem concerned not only the breakage, but above all the deformation that these suffered because of the differential retirement when, red-hot, were sprinkled with water in only a side.

In the United States the first fire-proof systems were experimented and had big diffusion; these systems foresaw the covering with uninfiammable and insulating materials, like stone or brick elements and above cement plasters.

The more diffused and convenient solution proved the covering with special brick pieces, solid or hollow, connected from steel little spanner and with external striped surface for allow a better plaster adherence.

For structural iron elements formally complexes, like the shaped sides or the staircases helices with a circular plant, it resulted more easy the covering with chalk or bastard mortar on a cement or hydraulic mortar floating coat. A double net layer, with a thin mesh in the inside and bigger in the outside, leave an empty interstice and guaranteed the plaster adherence. (Figure 3)

The fire damages revealed the not sufficient use of incombustible materials, or with a good fire resistance, but it was necessary that the same constructive system they constituted were really sure.

Besides, the iron and cast iron employ in the staircases construction allowed a reduction of the encumbrance and more complex formal solutions with small expense. The refined cast iron components, produced in a semi-industrial way, didn't require in fact ulterior decoration works, resulting therefore more convenient as regards the wood elements, that required entirely an handicraft workmanship. So, while the same cast iron elements, like the step tread and step elevation, characterized in a diffused way the small service staircases as well as the richer examples, the use of iron forged for model the staircases sides with a more articulated development and the iron beaten finishes employed in the more fanciful handrails were exclusive appanage of the last ones. These considerations underline like already to the half of XIX century, in a period of an industrial product exaltation, the handicrafts manufactured articles continue to be considered an «added value», above all for these small structures that were real furniture elements for the inside rooms, often characterizing also the buildings outside with their discreet or declared presence. (Figure 6,7)

The iron staircases success, above all in some contexts, it was favourite because these were presented by the sanitary engineering manuals and the hygiene treatises, that had wide diffusion

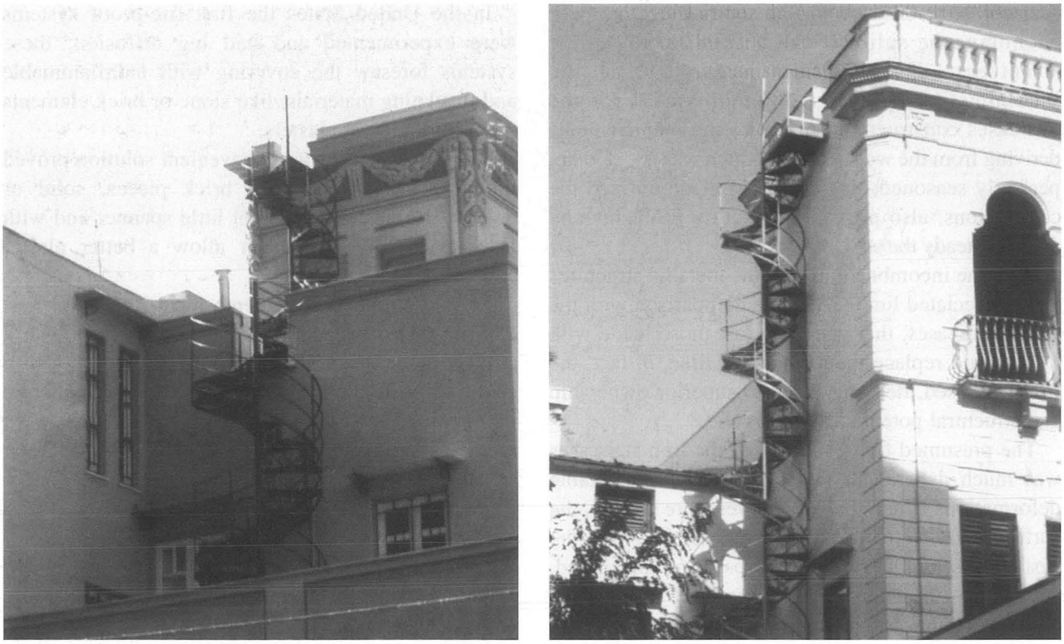

Figure 6, 7
Iron winding staircase of service, in the terrace of a building in Leopardi street, in Palermo; an other similar staircase in the Cavarretta villa in Palermo, designed by the architect Ignazio Greco (1910)

beginning from the second half of the XIX century, like structures also able to guarantee hygienic and salubriousness conditions. In fact these structures were often deprived of step elevation, allowed a better ventilation. Besides, the perfectly smooth and not porous surfaces facilited a more deepened cleaning and didn't allowed the micro organisms development and sprouting.

The step treads, despite result less hygienic, had often realized with wood or stone; the iron ones resulted in fact little sure because of their smooth for the use surfaces. It was preferable to employ cast iron perforated step treads or treads covered with materials easily washable like linoleum.

Constructive and technological characters

The constructive systems that characterized the first iron and cast iron staircases had often derived from those traditional systems of the stone or wood structures.

That explains why, still at the half of XIX century, in many cases for these materials has not matured neither an autonomous expressive form or, also having acquired an own structural affirmation, a connections system that didn't require an ulterior definition during the construction.

We refer to the staircases in which the static operation of the stone monolitic steps was re-proposed with cast iron steps melted only in an element; this was, for example, the case of the «at neck» (*«a collo»*) winding staircases, or the case of the bound-staircases (*«scale a sbalzo»*) with a rectilinear development.(Figure 8)

Solutions of this type were soon abandoned for a decomposition in single portions of elevation, tread and sides, allowing as a more easy assemblage, simplifying the fusion operations of the elements, guaranteeing a better adaptability to the different planimetric schemes and facilitating the movement and the transport, fact surely not negligible in a period

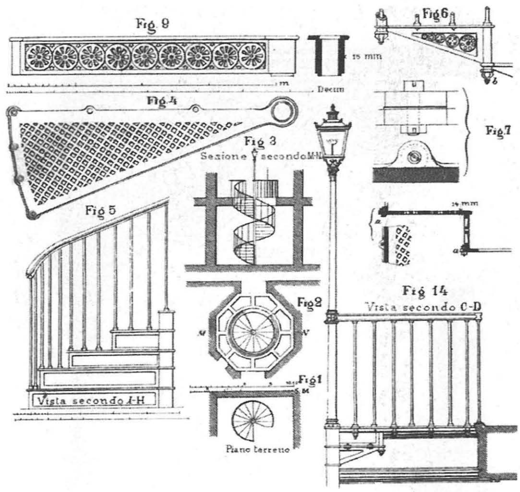

Figure 8
Cast iron winding staircase, with in an only piece melted steps of the Stoccarda station. (Breymann, vol.III, tav.69)

in which the metallic semi-worked elements represented an important export voice for Countries like England, Germany and above all France.

Examples of rectilinear cast iron staircases were very rare, either because they resulted excessively heavy, or because the very long rectilinear climbing elements were subjects to strong flexion solicitations to which the cast iron hardly could withstand.

The cast iron was used in prevalence for the realization of winding staircases with central mainmast, that could be constituted by the overlap of «glasses» jointly with the staircases, or that could present, in the case of soul constituted by an only piece, flanges, joint systems, rings or angle-iron applied, that they allowed a simple connection between staircases elevation and tread.

When the mainmast was represented by the «at cylinders» type with sleeve, (Figure 9) it was possible to insert inside it an iron bar, fixed to the foundation through a base plate, that arrived to the landing or that it could be prolonged up to the ceiling of the superior floor; for greater diameters it was possible to apply an oak upright. Also for avoid small mutual movements between the staircase parts, the «mainmast-iron bar» complex became solid by a cement-mortar throw, injected inside the soul.

The connection of the elements constituting the staircase had realized by the handrail upright that, spending through the two buttonholes to the external edges of two following treads and by the mediate sleeve jointly with the elevation, it made stable the whole structural complex. The connection between staircases elevation and tread had made still more rigid from the corbels insertion, that could be fixed by bolts connected with sweet-iron pins or through fused flanges. The iron corbels didn't develop only a static function, but they often were the elements that mostly qualified aesthetically the «at sight» staircases.

The sweet-iron pins had used in a diffused way like connection system, because the cast iron brittleness didn't allow the adoption of «at percussion» connections.

For circular plan of diameter better than 2mt, it was opted for «at empty spindle» *(«a fuso vuoto»)* winding staircases; in this case, the carrying elements, either that were the staircases treads or the elevations, they had inserted into the cage wall. An interesting example of this staircases type was represented by the staircase of the English Country House in Berlin. (Figure 10)

The cage diameter was equal to five times that of the spindle; the elevation, that constituted the carrying element, had thickened in correspondence of the inserted portion and acted from support for the tread. The two elements were connected by a squaring that contributed, with the big handrail batons, to the realization of an empty and very rigid spindle.

In this and in many other cases, the staircase was realized at completed building works, when straight the cage had stayed already plastered; that underlines the assemblage simplicity, effected simply with the use of a very sharp chisel that allowed to draw in the wall the buttonholes for the corbels placement. The assemblage facility (6 steps daily), the saving on the skilled workers job (they were sufficient a mason and a blacksmith) and the possibility of intervention in a not destructive way in existing buildings, favoured surely the diffusion of this type of structures.

For make lighter the cast iron staircases, in any cases, it was possible to not insert the elevations; the same staircase treads boring didn't absolve only an aesthetical function but she also contributed to a notable weight reduction.

The exclusive cast iron use for the staircases realization, initially very diffused for the facility with

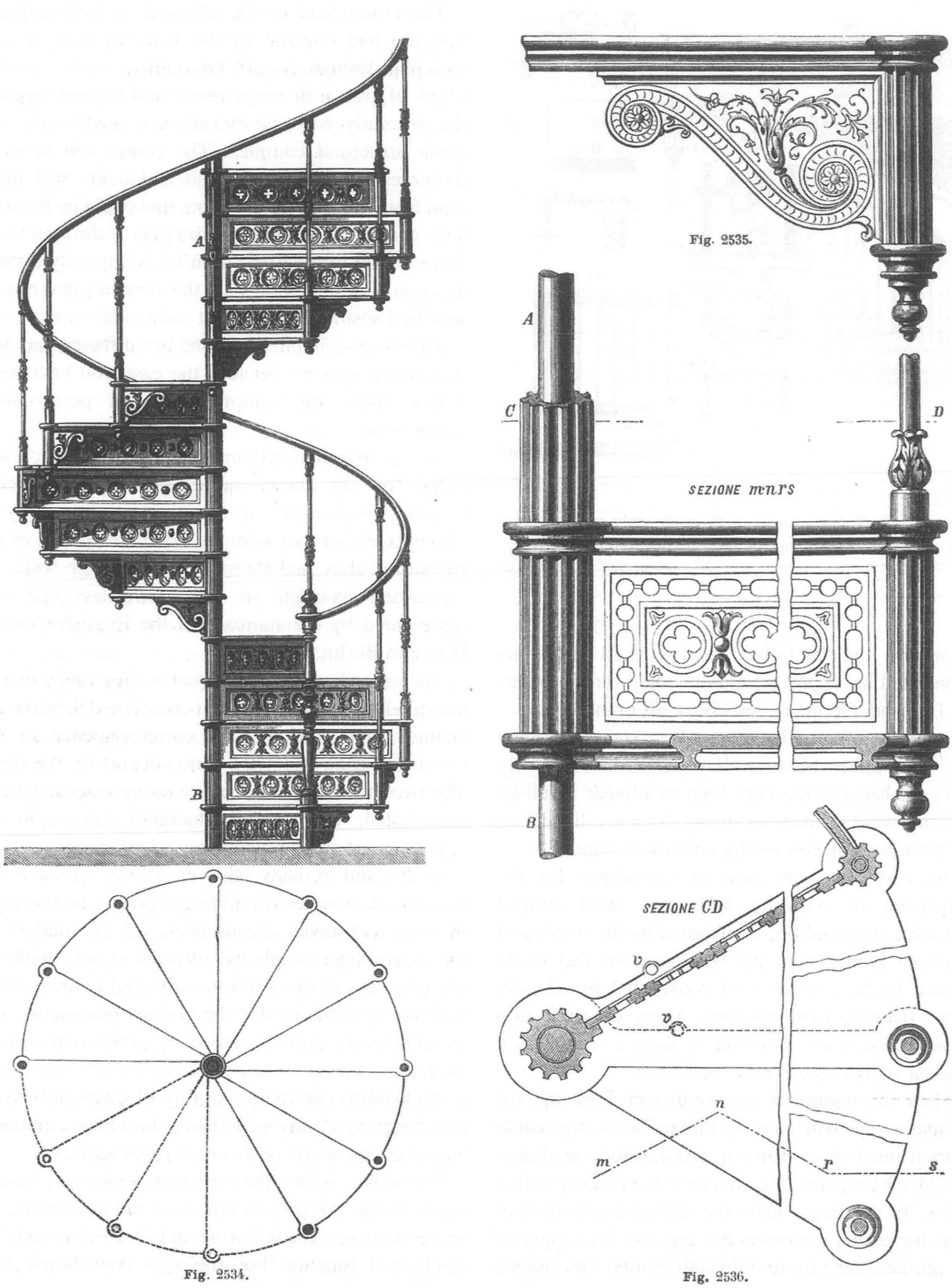

Figure 9
Cast iron staircase with assembled steps and sleeve-mainmast. (Pareto R., Sacheri G., vol.VI, p.I, p.1669)

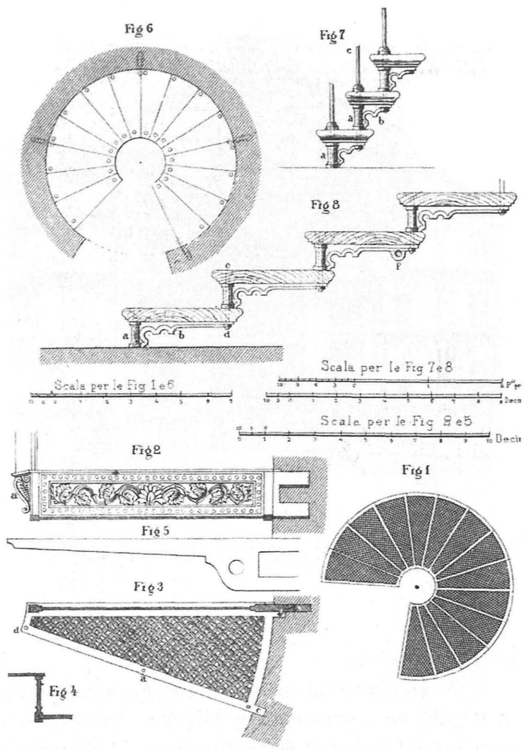

Figure 10
Winding staircase with empty spindle. (Breymann, vol. III, tav.71)

which were strained elegant elements also with reduced costs, was soon limited to the small internal winding-staircases, or rectilinear secondary staircases on which elevated loads must not transit. The facility with which the cast iron elements broke theirselves because of bumps or changes of temperature, it made them in fact less reliable as regards those realized by forged or laminate iron that guaranteed, at parity of resistance, an own weight reduction and a better safety, also in the case of the external staircases, mostly subject to the temperature variations.

However, the use of the iron laminates was limited to the structural parts because the smooth and naked surfaces was considered less elegant, and the beaten iron had used limitedly to the accessories like handrails and decorative elements, because of the elevated costs.

For this reason, the metallic staircases that had a better diffusion, were those in which the iron and cast iron potentialities could be exploited.

At difference of the cases till now illustrated, that take back constructive schemes re-leaded to the stone staircases, in the iron staircases the techniques of realization were inspired rather to the wood structures, because wood was a material similar to the iron for his good traction resistance.

The reference is evident in the so-called staircases with «at saddle» («*a sella*») steps, in which these last was supported by iron ribs («*costole*»), simple or double in horizontal or in vertical, in flat forged iron. The iron ribs, mails to the sides of the jaws («*branche*»), could be shape according to varied forms. Examples of staircases so conceived were the first one that located in the palace of the Alberto prince of Prussia in Berlin, in which the steps, melted in only a piece, were based both the extremities on two groins, and the second one that placed in German Cathedral always in Berlin; in this case the staircase tread had connected through iron squaring to a helicoidal groin. It was possible to realize either ramps of notable dimensions through some intermediary groins, or agile little staircases with iron steps or pegs. These last had realized inside or outside of towered buildings; for the easy construction and they for the reduced spaces that occupied, they were often used in the XIX century in the reconfiguration and splitting interventions of palaces and castles.

The ramps, constituted simply by flat irons, they were hooked to the intermediary landings that realized the walls connection by the means of an iron bar folded up to straight angle and inserted at the extremities. The stability was guaranteed besides by the presence of two corbels connected to the iron bar through brackets. In case of very long ramps, intermediary supports, constituted by wall-inserted corbels, were placed. (Figure 11)

These same staircases, for their lightness and for the structure slenderness that didn't hinder the light passage, had positioned in a stable way on the big skylights and greenhouses, for their maintenance and cleaning.

The sides groins could be replaced from laminates that they constituted the staircases full-sides. These could be realized with steps inserted in the sides, for architectures examples remarkable as regards their dimensions and decorations, or with superimposed

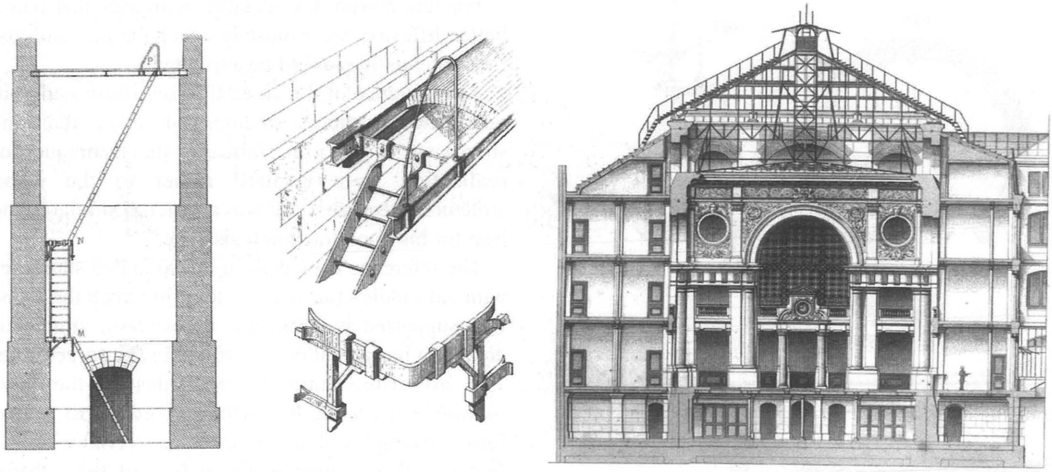

Figure 11
Service iron staircases, located in Paris skylights (Comptoir d'Escompte) and towers (Trocadero building). (*Revue generale de l'architecture*, 1884)

stairs, for the more common house-building. The staircase sides could be also realized, in the case of inserted steps, with **C**, double **T** irons or plates, hems with flat or angular irons. If the steps had superimposed the plate presented itself «rack-shaped» and could be formed in an only piece or by some parts, equal to that of the steps, connected through flat spiked irons.

The stairs were connected to the sides through **L** irons that followed the elevations and treads course; these last could be realized with stone, wood or striped or grained floor-plates. When one of the staircase sides of the ramp was adjacent to a wall, it was realized a step support through flat shaped irons provided of fasteners.

If through the designers ability and the blacksmiths skill that realized these hand-manufactered works, was possible to realize iron forged staircases presenting the more complex forms, at the same time the decoration problem of the smooth-plates surfaces constituting them was felt.

And still at the end of the XIX century the matter was so noticeable that one of the disadvantages attributed to the realization of this structures type was that, despite the staircase raw structure was expensive like a stone staircases, the exigence of superimpose to the naked iron a decorative apparatus made these ulteriorly onerous.

In the architectural manuals were intended models in which the ornamentation exigence had resolved with a proliferation of decorative irons, printed plates and of an endless variety of cast iron *rosette*, *brindilles* and *rubans*, what they rarely contributed to the staircase stability and thicker they instead constituted a notable load increase.

On the contrary, in the specialized magazines to the iron architectures was moved an only reproach, that they were uselessly complicated in the details and with forms too rich for the execution modalities employed. For these realizations was in fact evident that the iron demanded a better cleaning and clarity in the details, because some details, acceptable when hand-performed by stucco, stone or wood, they appeared heavy and artificial if reproduced using the cast iron. It was usually, however, already beginning from the second half of the XIX century, fluently and in a way just in that time definited like «commercial», the use of iron staircases cleverly assembled that resulted excellent examples for their structure, their disposition and economy, but in which rarely an adequate formal research could be founded. In the refusal for the undecorated surfaces we can realize as in that time the coeval were distant from formal solutions that could liberate theirselves by aesthetical

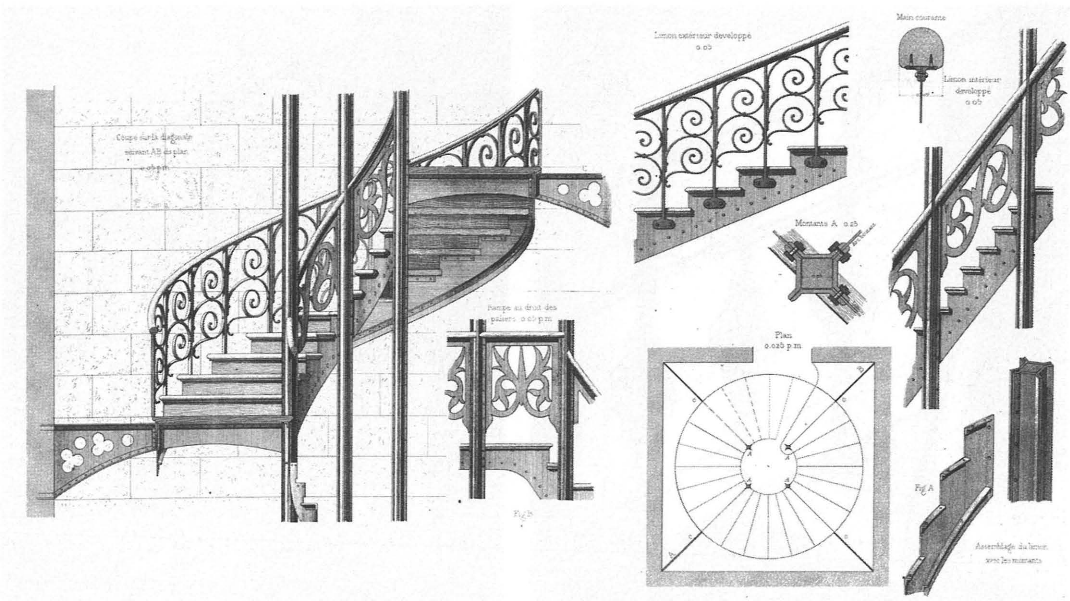

Figure 12
Iron staircase designed by the architect Viollet-le-Duc inside the Chateau of Eu

consolidated schemes and as any marginal realization of contemporary architets could result innovative, like these proposed also by Viollet-le-Duc.

In the 1876, at Chateau of Eu, he realized a staircase that for the forms study and the particular disposition represents an interesting case. (Figure 12)

The Viollet-le-Duc conviction that the employment of new materials like iron and a more rational vision of the structure and the distribution were the elements through which it would be expressed the XIX century architecture could also find its foundation in this small building work: no concession to decorative elements that they were not tightly functional, a very simple and clear structure conception, simple and visible details.

He built a staircase whose cage had arranged inside the old building walls, in which it was not possible insert the steps, besides the landings that allowed the access to the different floors didn't correspond vertically the ones to the others. The architect, choosing the simpler solution and, like we would say today, «less-invaded», planned an independent from the perimetrical walls staircase, that presented an empty nucleus definited by four uppercuts and externally delimited from a plates spiral, connected to the masonry through iron corbels, inserted in correspondence of the vain angles. The four vertical supports had constituted from a square iron soul contained between four angular, and between these was inserted the rack-plates. In correspondence of the rack-plate prominences and along the inferior border, small angular permit to fix the staircase treads and the intrados wood tread-covering. The two handrails, interior and external, they presented different solutions; the first one was realized by a cut-plate with phitomorf motives, the second one, in beaten iron, had constituted by square uppercuts and spiral motives.

In the same period in Palermo were realized iron and cast iron staircases that they could be definited semi-industrial and semi-handicraft for the components used, and for the skill with which they had adjusted to satisfy the designer exigences. We could consider two examples, the staircases with empty spindle in the Villa Whitaker-Malfitano veranda, designed By Ignazio and commissioned to the Izambert foundry in Paris in the 1882, and that of the Pignatelli Institute, located in the Colli plane. (Figure 13)

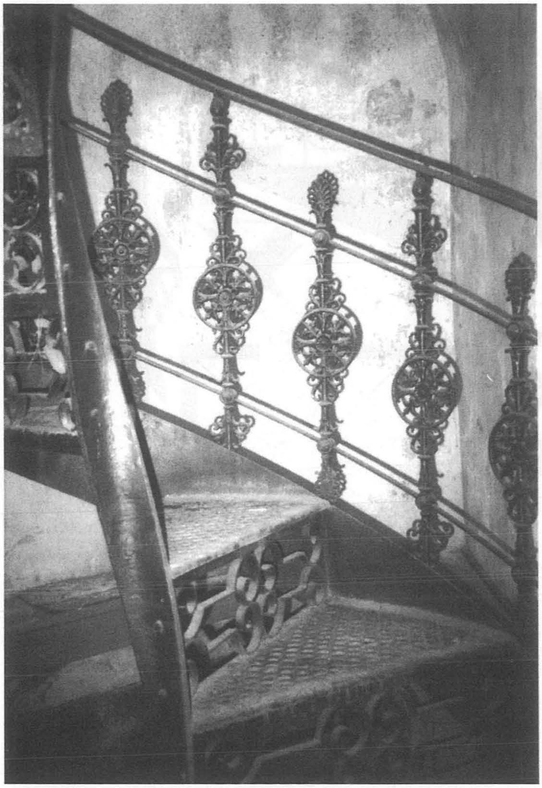

Figure 13
(On the left) Iron and cast iron staircase in the Malfitano Whitaker villa in Palermo; (on the right) winding staircase in the Pignatelli Institute in Palermo

A technique development that was accompanied to a taste evolution could be individualized in the replacement of full plate sides with iron frameworks that, reducing at minimum the iron employ, they made apparent the structural line like a decorative element.

Often in the iron frameworks staircases the handrail collaborated with the carrying structure because the upright also assumed the function of rods, making in this way the staircases sides extremely resistant.

A particularly interesting example for the structural clarity and for the light skeleton without decorations, it was that of the Venetian Cotton-mill staircase in Venice designed by the engineer Mazzucchelli, in which the superior **T** current was rectilinear, the inferior was curved and the connection rods had realized simply with flat irons. (Figure 14)

The staircases with iron framework sides evolved in simpler forms, based on the connection of a maximum number of two or three elements. This was the case of the Joly, Wilk and Puls staircases. These, constituted by parts of reduced dimensions, presented the advantage of be transportable, easily assembling and aestetically pleasant. (Figure 15)

Notes

(*) In this unitary study, it's possible to ascribe the first paragraph to ing. Tiziana Campisi, and the second paragraph to ing. Calogero Vinci.

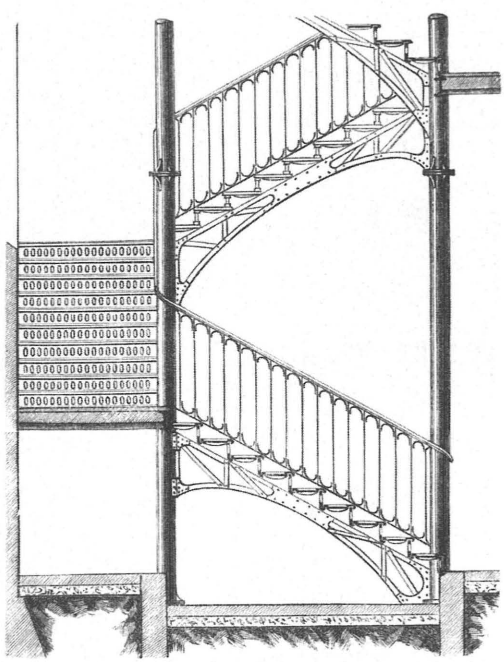

Figure 14
Staircase with iron framework sides, in the Venetian Cotton-mill. (Pareto R., Sacheri G., 1896)

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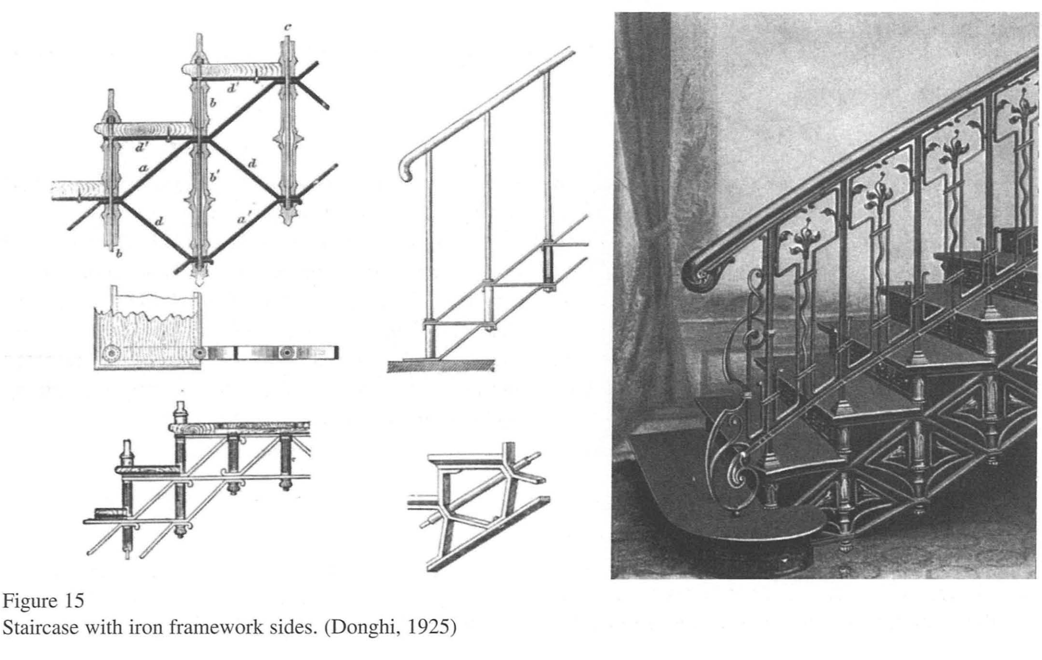

Figure 15
Staircase with iron framework sides. (Donghi, 1925)

On the origin of some «whiteness» carpentry rules

Angel L. Candelas Gutiérrez

In the greater part of Spain during the Middle Ages the construction of wooden roof structures used the system called *par y nudillo* (principal and collar beam), system which, with certain variations, was also used habitually in Northern Europe.

These wooden structures had already come into use by the 13th century, reaching their high point around the 15th century, after which the tradition went into decline. By the late 17th century, the loss of this building tradition had become notable, although roofs of this type were still being built in the 18th century.

The carpenters who executed these structures, and the works themselves, took the name of the light colour —white— of the wood, once peeled and sawed, and hence the term «whiteness carpentry».

We do not really know why this building tradition was lost. In reality, in architecture everything is subject to the changes in society in general, which have a direct influence on tastes and the adoption of different styles.

In the case of Spanish structural carpentry I believe that two circumstances combined to cause its progressive disappearance. On one hand, the absence of texts or manuscripts describing the technique and, on the other, the appearance of successive architecture treatises whose illustrations of roof framing generally reflect the legacy of the Roman tradition —the truss and purlin system. This system became prevalent in a short period of time, in the 17th century, both in Spain and in other European countries.

There are only three known texts about Spanish «whiteness carpentry», in which the configuration of these frames is described. All three appeared in the 17th century, by which time the tradition was already in decline; indeed, López de Arenas notes in his manuscript that his purpose in writing it was to slow its falling into disuse and make up for the lack of knowledge of it among master carpenters.

The aforesaid three texts are the manuscript by Diego López de Arenas *Primera y segunda parte de las reglas de la carpintería*, dated 1623; a part of the manuscript written by Fray Andrés de San Miguel, circa 1640; and the never published manuscript by Rodrigo Álvarez *Breve compendio de la carpintería y tratado de lo blanco . . .*, also from the mid-17th century.

It must be borne in mind that the knowledge required to execute major structures, many of them involving complex tying systems, was transmitted orally within the guild, and possibly with the prohibition of such knowledge being revealed to people outside the trade itself, as was the case in other construction guilds. This fact, apart from the possible illiteracy of the majority of the carpenters, may be the reason behind the inexistence of written texts from the peak period of this carpentry.

In executing these structures the carpenters followed a series of rules which only came to light after the publication in 1633 of the text by López de Arenas.

The interpretation of this text has entailed serious difficulties: it was attempted by researchers from the

field of history, i.e. M. Gómez Moreno, and from the engineering and mathematical fields, i.e. Prieto Vives, but it was not truly understood until the 80s, based on research by E. Nuere.

Now, drawing on E. Nuere's findings, we can finally interpret with absolute correction the variety of geometric constructions and the meaning of the text in the above treatises.

This article is intended to take things a step further, attempting to understand how the carpenters could arrive at some of the rules that appear in the treatises, which, while not the fruit of accident, nor did they correspond, evidently, to the application of a scientific or mathematical knowledge that the carpenters of the guild could hardly have possessed.

Very little is known about the way in which the rules used in framing carpentry came into being: the treatises simply explain them, and we still lack sufficient information regarding the frames built over the centuries to answer this question. It would seem logical that the carpenters arrived at these rules after a long evolutionary process in which they would have tried out numerous possibilities; then they would have stuck with those that combined correct structural behaviour with easy application.

Specifically, I shall refer to the origin of two of the many questions that appear in the treatises: on the one hand, the possible origin of a basic rule in the structural configuration: the position of the collar beam and, on the other hand, that of a geometric construction used to obtain the hip section.

In the first case we shall see, based upon structural verifications, that the rule results from the combination of an appropriate structural behaviour with ease of construction.

In the second case, we shall see that the geometric solution, in principle complex, is the result of the interpretation of certain circumstances that the carpenters were able to observe in the construction itself.

Hypothesis on the origin of the rules for determining the positioning and length of the collar beam

In traditional Spanish carpentry the most common position of the collar beam is at a third of the height, with which the roof projection generates lengths of a third of the span of the faces and another third in the centre —called the *almizate*.

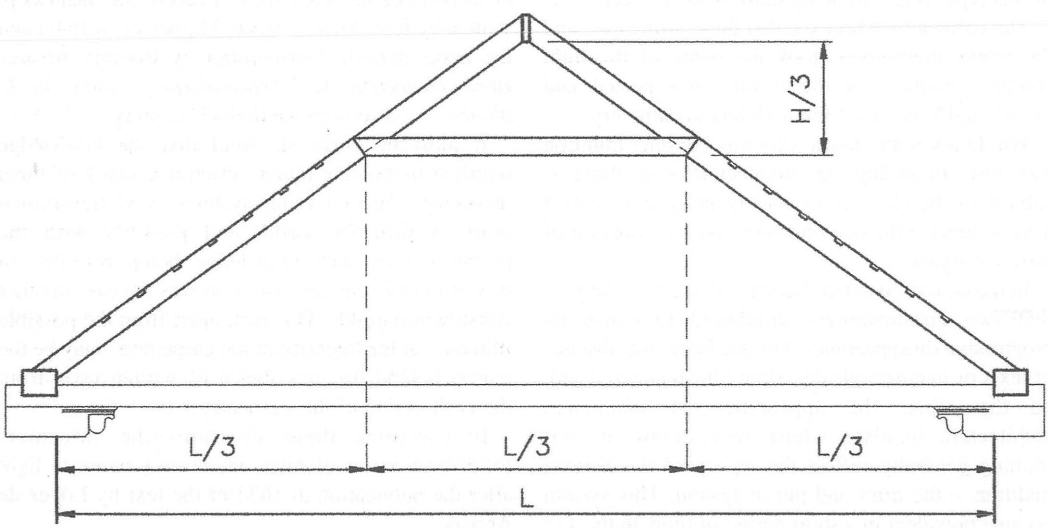

Figure 1
Most common geometric scheme in Spanish *par y nudillo* frames, similar to that described in the Treatises

In the cited treatises, all three authors indicate this position, although in exceptional circumstances, such as those related to ornamentation, other arrangements do occur.

On the other hand, the most common roof pitch is 36° —arrived at by dividing the semicircumference by 5— in keeping with the procedures used for the execution. Figure 1 shows schematically the simplified section of a hypothetical frame designed according to this rule.

First we let us look at questions of structural performance. In order to simulate the possible evolutionary process I have analysed the mechanical behaviour of structures with different placements of the collar beam, trying out those placements which most likely would have been used, and which come of a simple division of the length of the principal. Thus, I have taken into account the position of the collar beam at a third of the height —the most common— at half height and at the upper fourth of the principal.

Figure 2 shows the diagrams of bending moments and axial stress associated with these configurations. Without the need for more exhaustive calculations, any reader with a minimal technical background will be able to understand from these diagrams that the solution of the collar beam at a third is that which results in the most appropriate stress distribution. After observing the real behaviour of other solutions, the carpenters would have arrived at this very same conclusion.

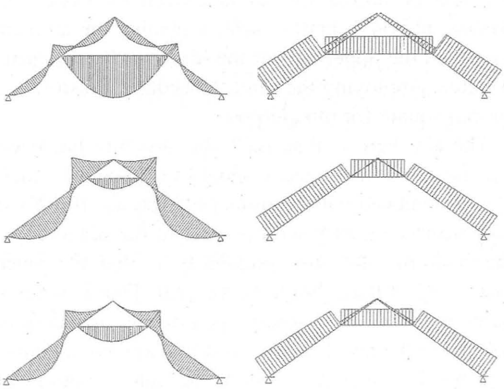

Figure 2
Diagrams of bending moments and axial stress in *par y nudillo* frames with different collar beam lengths (1/2 1/4 and 1/3 of the width)

Nonetheless, the formal characterisation of most of the constructions does not lie exclusively in correct structural behaviour. Without doubt other added circumstances must have come into play for this solution to have prevailed. Here is where the need to attain a regular arrangement comes in, both as an aesthetic recourse widely used throughout history and for the introduction of the ornamental *lacería*.[1]

In fact, in the sizing and distribution of the rafters within the frame the rule of thumb was to separate the rafters by a distance equivalent two times the thickness of the principal. On the other hand, that thickness was obtained precisely by dividing the width of the bay to be covered by a multiple of 3. The thickness thus obtained became the unit of measure used in the construction of the frame.

The plane formed by the collar beams —the *almizate*— is also where the rich and complex polygonal pattern of *lacería* characteristic of the Mudejar construction was habitually incorporated. The latter requires the existence of a regular arrangement that serves as a base for the geometric development of the ornamentation.

Evidently this regularity can be achieved in many ways, but the one used in Spanish framing carpentry, and referred to in the treatises, based on the use of dimensions multiples or submultiples of 3, has the advantage of easy application without the need to make calculations, complex auxiliary constructions, or even plans.

Indeed, the procedure described in the treatises combines correct structural behaviour with ease of construction. The set of the rules cited leads to the regular lines of the *almizate*, capable, on the other hand, of absorbing the horizontal loads on the frame, and providing the basis for laying out the ties.

On the basis of the above, I believe that the origin of the rules for the location of the collar beam and determination of the thickness of the timbers is perfectly in accordance with structural and formal circumstances. Indeed, in Spain, a system was consolidated in which two highly positive factors came together: on one hand, the great stability of the frames and, on the other, having found from a very early date an ornamentation process which offered great variety and richness without altering the basic structural system.[2]

Hypothesis on the origin of the geometric construction used to obtain the hip section

The other rule I shall address is that which served to obtain the cross-section of the «moamar» hips.

By way of introduction I should refer to a feature that distinguishes Spanish framing carpentry from that of other countries. The habitual procedure in almost all cultures for building a roof of various faces is fit a sloped timber on the edge delimiting one section from another —the hip. In Spain, in addition to this procedure another more refined one was employed, placing a hip on each of the planes that made up the roofs, thus on the edge there appear two adjacent hips. This is what is called the *«moamar»* hip, and it is associated with the prefabrication of the roof planes on the ground and the subsequent fitting of the completely finished spans. Although this is not the subject of this article, it may be said that it is one of the first examples of large-scale prefabrication in the history of construction.

In the definition of the moamar hips, the whiteness carpenters achieved great subtlety and perfection, seeking the correct visual effect. In fact, if we build the hips with rectangular-section timbers, observing the area between them we see that the inner faces of the hips are not parallel, due to different slopes of the spans. Our impression is that they are angled inwards with respect to the fictitious plane formed by the inner edges of the hips. In order to correct this effect, a trapezoidal hip section, known as a *campaneo*, emerged. The three writers explain how to obtain this trapezoidal section in moamar hips. The construction method following is from R. Álvarez:

> De como haras la esquadra De limas fairas . . . Para Sacar la esquadra De lineas fairas se a de tomar un pedazo de madero labrado al marco, que huviere de Yr la madera para la tal obra. Y hecharas en tal pedazo De madero un trazo con la caveza De la Planta Pitagorica, que los Arquitectos llaman Cartabon de aquatro, en el ancho o tabla; Y echaras otro trazo con la caveza del cartabon De Armadura que junte con el de aquatro, Y luego Daras la Buelta al madero por el canto, y adonde fenezen los trazos que Yciste en elaz que fueron a parar al canto hecharleas Dos trazos con la cola del alvanecar a que Armaren la tal obra, ora sea cuadrada, ora ochavada: y el claro o cantidad que ay de cola a cola del alvanecar la tomaras en un compas, y por la esquina del madero pondras la tal medida poniendo la una Punta del conpas en trazo quadrado y con la otra aras un Punto adonde llegare que sera entre el trazo de la caveza De la Armadura y el quadrado, y este viage que haze este trazo Con la esquina Del madero es la esquadra de lineas fairas como lo Muestra su figura. La letra A. corte quadrado la B. caveza de armadura la C. y D. las colas del alvanecar la E. el desvio de las dos colas la F la horma de la esquadra. (Álvarez, R. 16??, 39v- 40).3

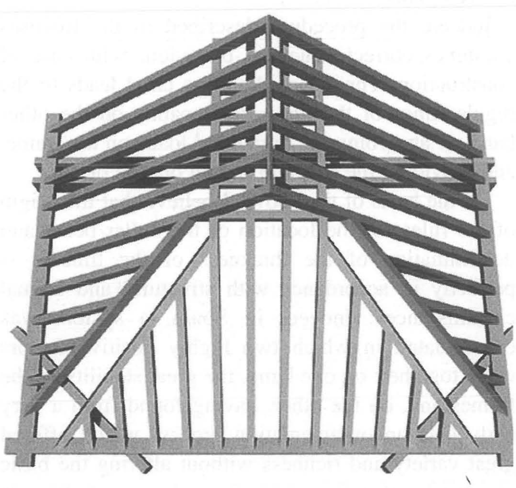

Figure 3
Spanish «par y nudillo» roof frame. Note the double *moamar* hips in the foreground

The procedure that Rodrigo Álvarez describes in this text is similar to that employed by López de Arenas, although better. Arenas obtains the amount by which the upper part of the hip must be widened. Álvarez, employing the same procedure, constructs a special square for this purpose.

The aim here is that, once the structure has been executed, the inner faces should be parallel, which can be achieved with a number of systems. But if we also want symmetry with respect to the diagonal of the building, the only possibility is that the inner faces, once fitted, should be vertical. That is what is achieved, with absolute geometric perfection, following the process described by the three authors.

Here I will not go into the interpretation of geometric construction resulting from the cited paragraph, question which, on the other hand, was addressed by Nuere (1985), though using the description by López de Arenas.

Once this interpretation is known, it is relatively easy to achieve the arrangement described in the treatises, but what is not so evident is how the carpenters arrived at this rule. Researching this question, I have formulated a hypothesis based on the geometry that the carpenters could observe in their first attempts to achieve parallel inner faces on the moamar hips.

Figure 4 shows the how a moamar hip joins the first principal of the structure. The left-hand image shows a rectangular-section hip and, at the right is the trapezoidal section sought by the carpenters.

In my opinion, some carpenter must have realised that in order to achieve vertical inner hip faces he had draw the lines from the edge of the joint between hip and principal with a *cabeza de la armadura*[4].

With this line, they could then draw on the face of the principal the sloped section of the hip they wanted to use, as we can see at the left of figure 4. However, what they needed was the straight section of such a hip. Nowadays this question might seem to have a simple solution; in reality it is no more than the folding of a section over a plane. But at the time they did not even work with plans, nor did they have knowledge of descriptive geometry and, as occurred in the formulation of other rules, they had to use the construction elements themselves as a basis for measurements and work processes.

I imagine that what some clever carpenter would have observed would be something similar to that shown in figure 5, where we see a plan view of the joining of a principal and a hip (next to which is the projection of the joining of a hip and principal).

Effectively, the carpenter, standing on the frame, could see that in the sloped section the added distance A-B could be obtained with «Cartabon de aquatro, en el ancho o tabla; y echaras otro trazo con la caveza del cartabon De Armadura que junte con el de aquatro».[5] And he might also realise that drawing two lines parallel to the face of the hip —achieved with the angle defined by the set square known as an *albanecar*— he could obtain the real dimension of the upper face of the hip.

All that remains is to calculate the distance between the two lines B-D in order to obtain the thickness of the timber from which he had to start in order to then cut the sloped face, and hence be able to prefabricate the hip with absolute precision before fitting.

On the other hand, Rodrigo Álvarez (16??) includes in chapter 43, dedicated to obtaining the campaneo of the hip, two drawings (fig. 6). The first

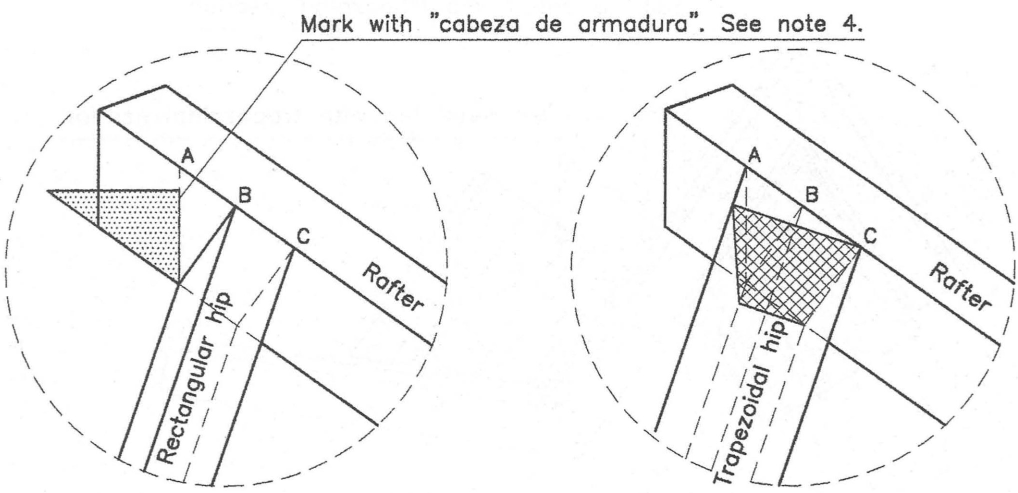

Figure 4
Joining of a *moamar* hip to the first principal of the structure, using straight-section timbers (left) and with the sought-after trapezoidal section (right).

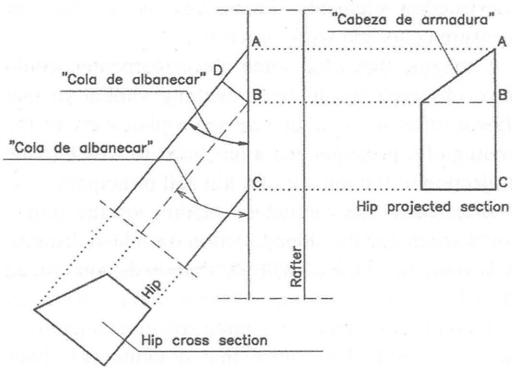

Figure 5
Plan view of the joint between hip and principal

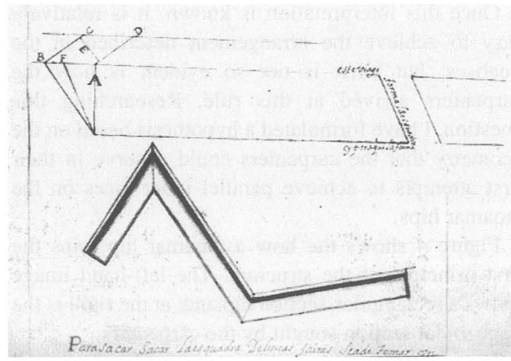

Figure 6
Drawings that appear in chapter 43 of the manuscript by R. Álvarez

is the graphic construction of the process he describes in the text cited above and to which the author himself refers. The text does not mention the dotted line which, as a second drawing, appears under the previous construction. That aroused my curiosity; convinced that this line must be in some way related to the campaneo of the hips, I did a series of geometric verifications based on the original manuscript and found that the angles formed between the lines in the drawing are precisely the angles, acute and obtuse, (fig. 7), that Álvarez obtains for the campaneo of the hip in the first drawing.

Thus, I believe that what Álvarez intended with the dotted line in the drawing in chapter 43 of his manuscript is a tool, a square, which would be built for each frame, with objective of facilitating the

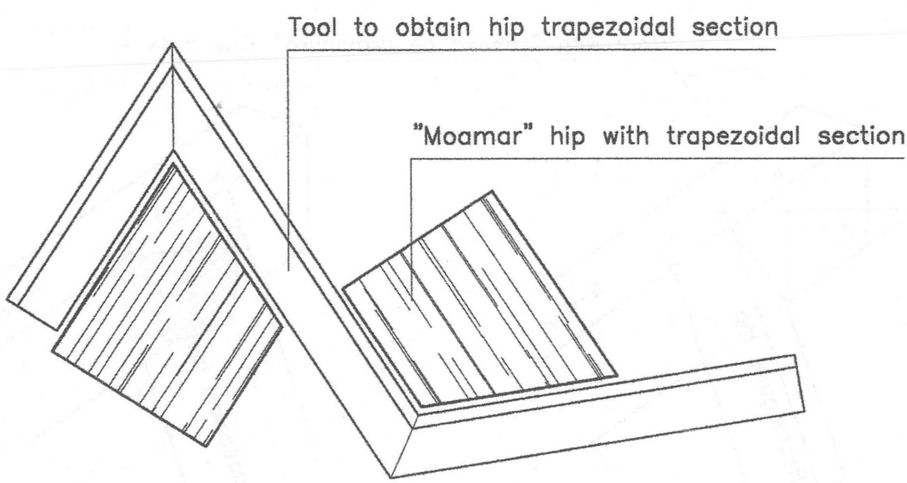

Figure 7
Interpretation of the lower drawing in figure 6: the dotted line drawn by R. Álvarez in chapter 43 could be a square for aligning the campaneo of hips. The angles of the dotted line coincide with those which define the bell in the upper figure. It is, therefore, a tool

execution of the hip section. This square would be placed at regular intervals to verify and obtain the uniformity of the section throughout the piece. Perhaps it is to this that Álvarez refers when, at the margin of the drawing, he writes: *«this line forms the square // and here the form»*.

Or comparative purposes, I wanted to reproduce the geometric construction which for the same objective —obtaining the campaneo of the hip— López de Arenas includes in his manuscript of 1619 (fig. 8). Expressing himself in different terms and with a different graphic representation, he obtained the same hip dimension as in the cited text by Rodrigo Álvarez.

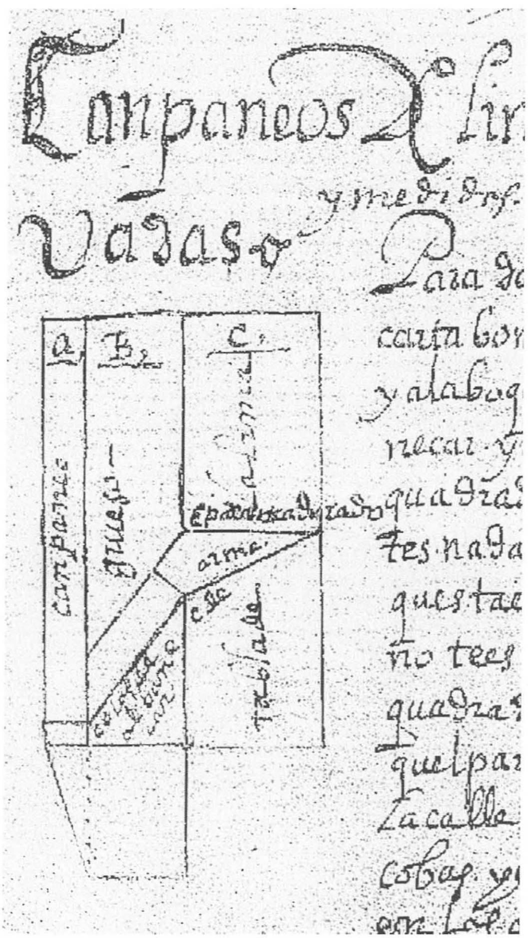

Figure 8
Construction by López de Arenas (1619) for obtaining the campaneo of the hip. Page 7v of the manuscript

These differences in expression lead me to think that Álvarez did not come to know this process through the texts by López de Arenas, and that he had certain experience in the execution of frames with moamar hips. This fact, along with other similar verifications, demonstrate the originality of the text by Álvarez, although it includes fragments of text copied literally from the treatise by López de Arenas.

Notes

1. The Spanish word for geometrical decoration of straight lines forming intersecting polygons and star shapes. Inherited as a decorative motif from Moorish sources, it was much used by Mudéjar craftsmen in Spain and Portugal.
2. This situation is rather unique in European carpentry. The greater pitch of the roofs in central Europe, determined by climatic conditions, leads to significant stability problems, and thus the great variety constructional arrangements that appeared outside Spain. Spanish carpenters were able to dedicate their efforts to perfecting the symbiosis between structure and ornamentation.
3. This text is difficult to understand in Spanish, both for the terminology and the syntax. It is thus practically untranslatable.
4. Drafting instrument: set square with an angle equal to the pitch of the roof.
5. Which might be translated as: «Mark a 90º line on the thickness, and another with slope of the frame»: the resulting triangle is the increase in section of the hip projected on the principal».

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The constructive techniques of the moorish roofing frameworks: The case of the Mirador of the Reales Alcázares of the Catholic Monarchs in Seville

Cecilia Cañas Palop

In recent times the roofing frameworks which cover the Mirador (observation pavillion or covered area) atop the Reales Alcázares (Royal Palace) in Seville have been restored. Thanks to this restoration it has been possible to examine first-hand and close-up this magnificent example of Moorish architecture, constructed during the reign of the Catholic Monarchs (Ferdinand and Isabella), which transport us from today to a very special epoch, and which speak to us of six long centuries that have left their mark on each of the basic elements.

The Mirador of the Catholic Monarchs is located on the top of the high Palace of Don Pedro above what is today the Hall of King Carlos V. Its construction caused a partial destruction of the upper room of the prince, which today is called Don Pedro's Bedchamber, cutting off the arched entryway to that room, due to the difference in height between the two. It is open on the one side looking over the garden and on the other to the high gallery of the Patio de Las Doncellas, or Maidens' Patio, and it is located between the aforementioned Bedchamber of Don Pedro and the Family Dining Room. According to Ana María Fidalgo, we had no knowledge of it until the year 1977, when the architect Rafael Manzano Martos discovered it during the restoration process being performed on the Palace of Don Pedro.

This same author (Ana María Fidalgo) asserts that during these recent restorations, the ceilings were returned to their original state, since they had undergone multiple remodeling projects up until then. Nevertheless, this is not entirely the case, since in more recent investigation and studies carried out by Inmaculada Ramírez López as the Restorer of the roofing framework, paintings and inscriptions have been discovered that had been unknown of to date, and that could very well belong to the original roofing framework.

The first restoration of which we have information is the one carried out by Juan Fernández and Juan de Simancas, since in a report given by the Masters of the reconstruction, there is an indication that in the new room above the garden, all the deteriorated wood of the roofing had to be repaired, being substituted by new wood and then being re-roofed.[1] Posterior restorations of the roofing framework have been numerous but not much information exists about them. However, they are visible in the multitude of elements that have been added throughout the centuries, with better or worse results.

A brief presentation of the framework

Its form is a square framework made up of four aguas and three roof slopes. Due to the constructive solution of the pares, we are analysing a framework of par and nudillo which uses limas mohamares[2] in the intersection of the different roof slopes. All these have been made using the apeinazado technique, and the final decoration was resolved with a tie-of-eight and mocárabe squares. The four faldones were

decorated alternating rows of eight-pointed stars and little ties.

Elements and method of construction

In order to describe the method of construction of the roofing/ceiling, the study was based upon an exhaustive, detailed analysis of each and every basic element and the relation between each element and the others.

In the first place, we present and place each of the elements of the roofing framework, beginning with those elements that have to guarantee its placement above the walls on which it rests. This perhaps is the least known area, because of being hidden from sight, and yet it is also the most important factor in guaranteeing the general stability of the structure. We will see later on how the greatest part of the problems that the framework has, have their origin in the supportive elements.

The hidden structure. The foundation of the framework

A very important fact worth bringing to light in regards to the foundations of this roofing framework, and also a few other roofings analysed (such as the Antecapilla or the Bedchamber of Don Pedro), is the lack of basic elements to guarantee its stability in the place upon which it rests.

The Mirador framework does not have elements such as *nudillos* or *solera*. The *estribos*, which are basic elements of structural stability, rest directly atop the walls of the room, without the minimum presence of some other element in the form of a layer of cement between the estribos and the wall, to somehow ensure the plane level of the receiving element.

We will differentiate between the transversal and longitudinal estribos, since there have been substantial differences detected between them in their geometric, supportive and conservational characteristics. The former are shorter and of greater edge than their perpendicular counterparts, and they are partially embedded into the wall, which makes it impossible to know the geometric characteristic of its total thickness; whereas the longitudinal estribos sit directly atop the crowning of the wall.

The longitudinal estribos are formed by two others, united in a lengthening to approximately the point of junction of the braces (tirantes). The length of the room (14 metres) is too long to support one sole element in the crowning of the wall. If we consult the «Ordenanzas de Sevilla» we see how long the different beams of wood had to be as a general rule, and these oscillate between the 25 feet of the *viga de acarro* (7,5 m.) and the 12 feet of the *medio ponton* (3,60 m.). So the longest beam of the Mirador's framework (9,13 m.) is quite a bit longer than the vigas de acarro specified in the «Ordenanzas»; and the rest, 4,9 and 5,2 metres, are located between the *media viga* (4,2–4,5 m.) and the *terciada* (5,32–5,70 m.) specified in the same text.

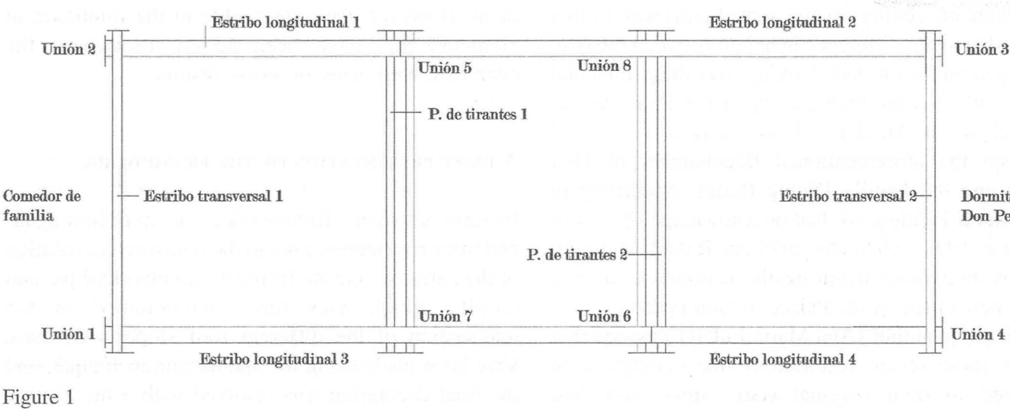

Figure 1
Location of the estribos and the tirantes in the framework

In the corners, the estribos belonging to different roof slopes formalise their union halfway up the wood, having a small notch which enables both pieces to not get displaced by the effect of the traction of the rest of the structures; nevertheless the difference of the section of the estribos makes it not possible for the longitudinal elements to have a cut halfway up the section. The junction of the estribos in their prolongation does not work as such. The elements which come together on them work independently.

Another important element in this section which came into being basically for structural necessity, is the tirante (brace). The size of the room makes these elements absolutely necessary. When the relation between the height and the width of the room is reduced, there is a danger of collapse in the crowning of the walls, since the horizontal pressure upon the estribos is very pronounced. The tirante collaborates in the absorption of part of the pressure and therefore gives equilibrium to the structure. A total of two pairs of tirantes have been provided in the length of the structure upon which the estribos sit.

The perimetral wall of the room, upon which the framework sits, is crowned by the estribo, as we have already described, which rests directly upon it; therefore since there is neither solera nor canes, the tirantes have to be embedded into the walls. The embedding measures about 22 cm.

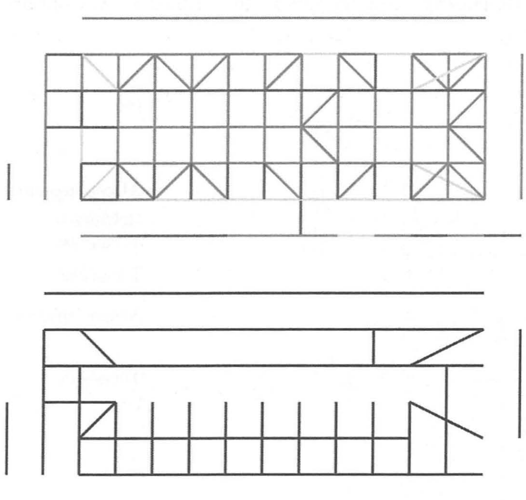

Figure 2
Embedding of the tirante into the wall and the union with the longitudinal estribo

If we stop to observe the junction with the estribo, we see that a 7 cm tall cajeado has been performed in the tirante, leaving a 2 cm tall espiga (see figure 2) to be placed into the estribo. We do not know exactly how much of this union is embedded into the wall; however constructive logic and the load that the tirantes have to absorb lead us to think that it must be as visualised in the representation.

At this time it is necessary to pause and make the observation that this is one of the points which has brought up more questions and doubts in regards to the analysis of the origins of the tirantes, since most of the data analysed lead one to think that the ones we see now do not pertain to the original framework but to the reformations carried out in 1908 in which the need to replace four wooden tirantes, replacing the originals, was noted.

> «To reinforce the wooden bracing, put into place four new wooden braces instead of the defective ones and to zancar their alfardas». (from the «Budget Book»)

The tirantes in a framework of a certain size or importance, have always been ideal elements for carrying out diverse decorative craftings, mainly because of their position, up front in the visual plane of the spectator. And, what is more, there has always been a decorative hypothesis about the use of double tirantes, since they made a good support for the ties of lacework (lacería) which covered the framework. It does not seem logical that in a framework of this kind, the finishing would be so rough and undelicate.

All of this data, together with those derived from the embedding into the wall and the junctions with the rest of the elements, bring one to consider that we are seeing elements that date back to the beginning of the 20th Century, new tirantes without any decoration (unlike the deteriorated originals which possibly had a tied crossover lacing effect, according to the standards of the epoch).

This doubt gathers force when we analyse the pictorial decoration which covers the sides of the tirantes, since according to the criteria of the modern restoration experts, these belong to periods prior to the reformation of 1908; to be exact, they are similar to others located in the frieze of the framework which

date back to the 16th century. Let us leave our «doubt» open then to the posterior development of the research being done in this field.

Finally, we will describe the part of the framework setting that is not hidden, which has the assigned job precisely to leave «unseen»behind them, each of the previously mentioned structural elements, and to serve as a pictorial support and visual transition between the wall and the framework of the roofing/ceiling.

The visible elements

The superficial elements of the *arrocabe* of the Mirador's framework are formed by a double alicer, in which the first plank is nailed into a mold or tocadura, which is also nailed onto a type of wooden bracket which becomes the perimetral zuncho at the sides of the room. This first plank, or lower alicer, is finished at the top with another one which serves as the mold or tocadura for the second one, right in the bottom coat of the brace. Behind this second alicer is hidden the junction between the tirante and the estribos with the wall, as well as the bottom part of the pares where they meet the estribo. The former estribo merely hides the crowning of the perimetral wall.

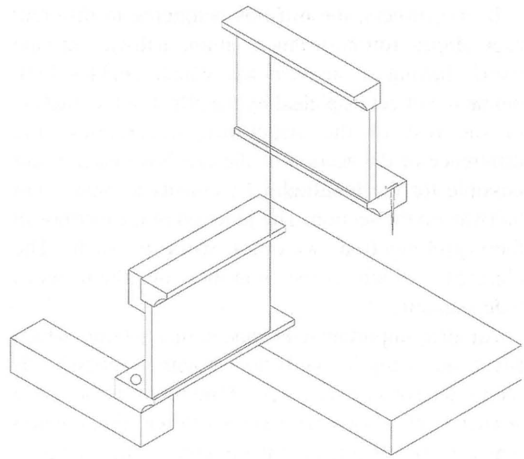

Figure 4
Union at the corner of the bottom parts of the planks (aliceres)

The inclined roof slopes

Before beginning an analysis of each of the constituting elements of the roof slopes, it is necessary to make mention of the set-square of the framework used (the inclination of the alfardas). Measuring directly upon the alfardas we obtain

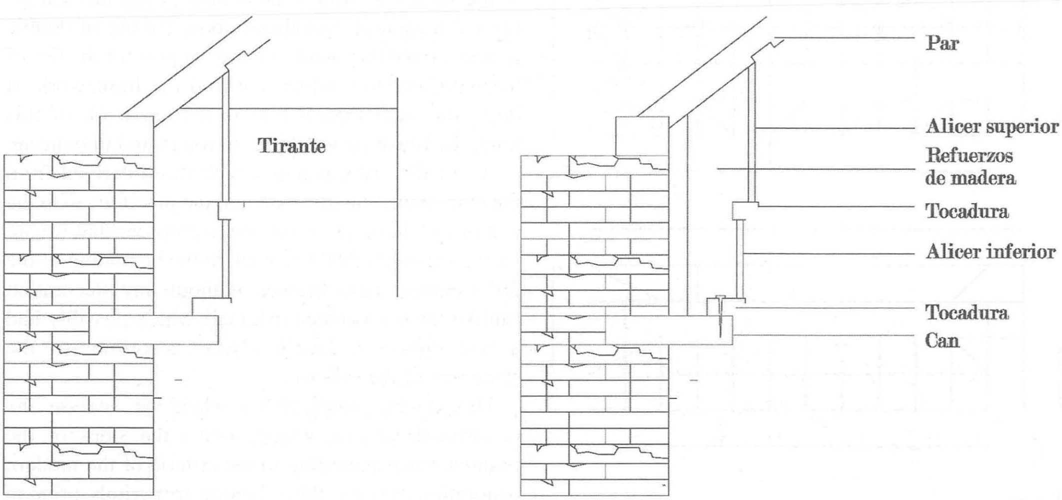

Figure 3
Section of the arrocabe

figures that range from 35,8 to 42.67 degrees. This fact confirms that the deformation of the structure is significant, since the theoretical set-square of the framework is at 37,6 degrees.

Structural composition

In the first place, as definitory elements of the inclined roof slopes, we must analyse the *pares* or *alfardas* and the *péndolas* or *manguetas* which necessarily appear in the frameworks that have limas. There are a total of six pares in the transversal roof slopes and forty-four in the longitudinal ones, plus a total of five manguetas which accede to each of the limas that pertain to the faldones. All of the pares are «live» since they form part of the lacework of the framework.

Since this is a framework of mohamar lima, each of the faldones has two *limas* at its lateral limit, with two from each of the contiguous faldones coming together at the corners.

Let us recall one of the fundamental rules for the procedure of making this type of structures upon which posteriorly tied lacework and stars will be crafted; the rule denominated by López de Arenas the «Law of *calle y cuerda* «(aisle and cord).[3]

In the case of the Mirador, the planning of the ties is precise, nevertheless the thickness of both pares and nudillos is of 7,5 cm, whereas the width of the aisles vary: in the testeros of the aisles they measure from 16 to 16,5 cm, and in the rest of the framework they measure from 14,75 to 17,5 cm (the majority being between 16 to 16,5 cm).

This statistic reveals to us that the differences measured are due to the movement experienced by the framework. It is produced more intensely by the longitudinal faldones, not so in the testeros, obviously the deformation increases in the extent that the length also increases. We can affirm that the real separation is close to 16 centimetres. We are speaking, then, of margins of error of around 0,25 and 0,5 cm, which in wood are not that significant, and even less so given the procedures used in that epoch.

This difference can be due to several different factors : on the one hand, the movements that the structure has had to support in general have been important, whereby the spaces between the alfardas could have grown (as we will later see, many of the pieces have become unjointed) and also, the passage of time can cause minor losses of parts of the wood. All this together with the assumption of possible small errors in the procedures of its construction make us maintain the validity of the aforementioned rule, to a certain extent at least. In any case, we can find no explanation that justifies making the aisle

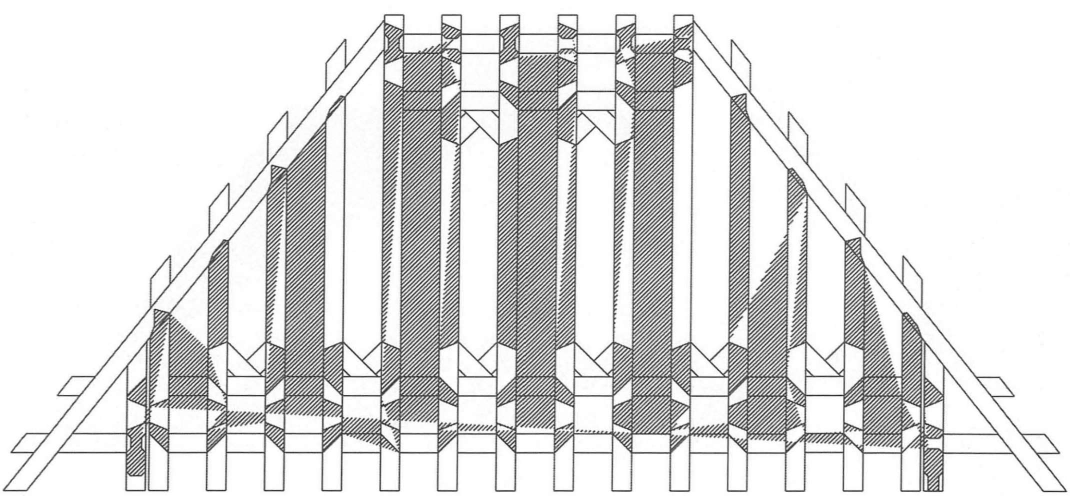

Figure 5
Location and cut of the pares and the manguetas

(calle) 0,5 to 1,0 cm wider, measurements that are insignificant. After having made this important clarification, we now will discuss how the different junctions between the aforementioned elements are formalised.

The junctions in the estribos of the pares and manguetas are formalised from the patillas to the barbillas, with the perpendicular cut done on the horizontal plane. Usually in order to fix the element securely onto the estribo this type of union was reinforced by nails, however in this case, no reinforcement has been has been detected whatsoever. The patilla measures 3,5 cm and the barbilla 10 cm. The cut of the barbilla is done at approximately 1/3 of the height of the par, conforming to what was the standard of carpentry. The limas have the same kind of union, however their state of conservation does not permit us to precise their geometrical characteristics.

It is also usual to find in this kind of structure that the pares culminate in the hilera which closes or culminates the framework at the top. The original roofing of the Mirador did not have an hilera and the pares were united by means of ensemble of garganta and quijera. This type of ensemble diminishes the resistent section of the pares at this point, therefore it is usually reinforced by means of some metallic element. But the junctions analysed lack any type of reinforcement, thereby they are in a very weakened condition.

On the other hand, the manguetas reach the limas at a cut perfectly parallel to the contact surface,and are joined by nails to the main structure. The limas are joined to the pares by the same means.

The rest of the structural elements pertaining to the faldones are the *peinazos* and the *arrocabas,*[4] located in respect to the pares and limas as seen in figure 6.

We will differentiate between the orthogonal peinazos and the inclined ones. The former are the ones that are located perpindicular to the pares. The junction with the pares is produced almost certainly by means of «*a romo y agudo*» with an espiga or wooden nail. Although this method has not been officially confirmed, in some of the pares the boxed space formed for the insertion of the espiga from its corresponding peinazo has been observed. The inclined peinazos are fixed both to the pares and to each other by nails. The junction of the arrocabas and limas is also done this way.

Non-structural Elements

In Figure 7 each and every element pertaining to the inclined roof slopes is shown, both structural and non-structural, which has the job of conforming definitvely the decorative layout of the framework.

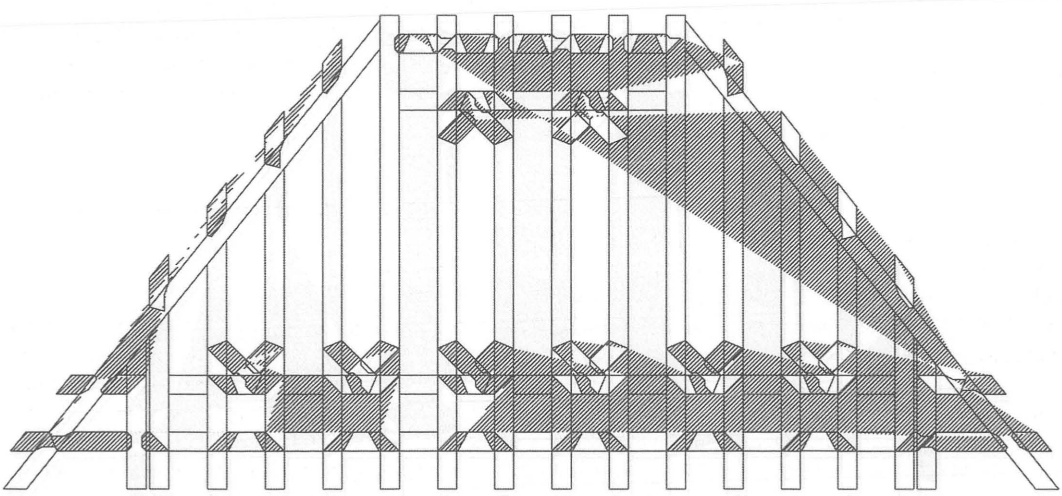

Figure 6
Situation and cut of the peinazos and arrocabas

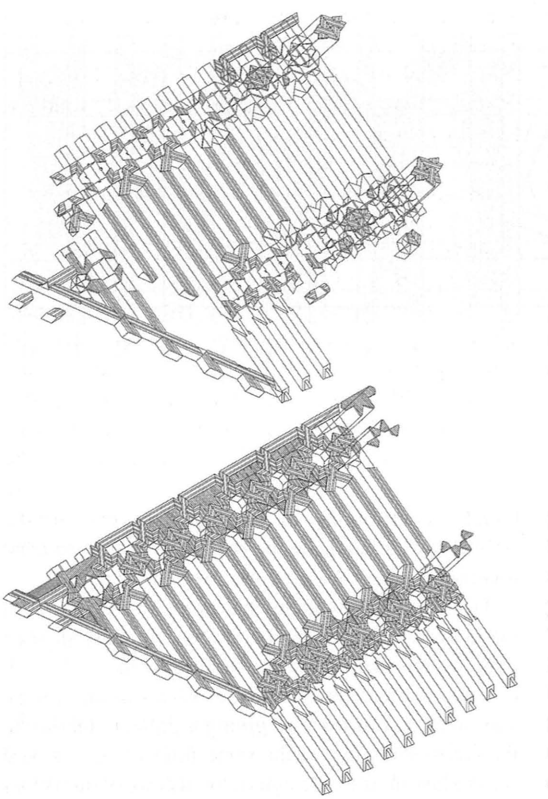

Figure 7
In the upper part the resistent structure and its respective unions are indicated, as well as the **taujeles** and the bases where the supports, wedges, and surface boards all appear. In the lower part of the figure, the inclined roof slope is shown with all of its elements in position, including the «tablazón de relleno»

The taujeles form part of the decorative layout of the lacework ties of the framework, without belonging to the structural plot. They are panels of 1,5 cm thickness which complete the layout, usually nailed to a *tablazón superficial*, which appear at each of the aisles contiguous to the eight-pointed stars. We must mention the union of these panels into the structure as a special characteristic. The panel has the same thickness as the rebaje (diminishing) done on the pares and the peinazos, coinciding with four protrusiones carried out upon the panel, so that they coincide and fit together perfectly. In the pares the protrusions are trapezoidal in shape, thereby increasing the supportive base.

There are also three distinct types of wedges that are nailed to the structural elements to fulfill basically a decorative mission. A first group forms part of the layout of the eight-pointed stars of the framework. Those of a second group serve as a base to support some of the taujeles, and a third group of wedges are placed in the upper part of the roof slopes, nailed to the peinazos, making the visual bypass from the inclined roof slopes and the almizate, in such a way that the eight-pointed stars located in these points, and the other decorative elements that are necessarily broken, all have their faces parallel. Instead of making the angle by doing a direct cut into the corresponding peinazos, this type of wedge is just added on.

There is another type of panel that directly forms part of the decorative figures of the framework called the *tablazón de relleno* (filler panel). There are areas on the faldones that require the location of these panels, which hide behind them part of the structure, which otherwise would be seen. These panels are an important part of the decoration of the framework since they are situated in the plane that is nearest in sight (on the same plane as the main pieces), and sometimes occupy a great part of the surface serving as a base for perfected pictorial decoration.

And lastly, we have the *tablazón del trasdós*. It is part of the fianl covering of the framework and its presence is taken advantage of to give a pictorial background to the framework. The panels are nailed to the resistent structure of the framework, pares and limas, and cover practically the entire exterior surface of it.

The plans of the nudillos. The almizate

Structural composition

In the first place, we shall locate the definitive elements of these plans, which are the *nudillos*. We must mark a distinction between two different types: the long nudillos, which are devloped along the almizate (types A,B,D and F), and the short nudillos, which are interrupted by different peinazos, which are necessary for the configuration of crossing and tieing of the lacing (types C and G). All of these are situated below the terciary.

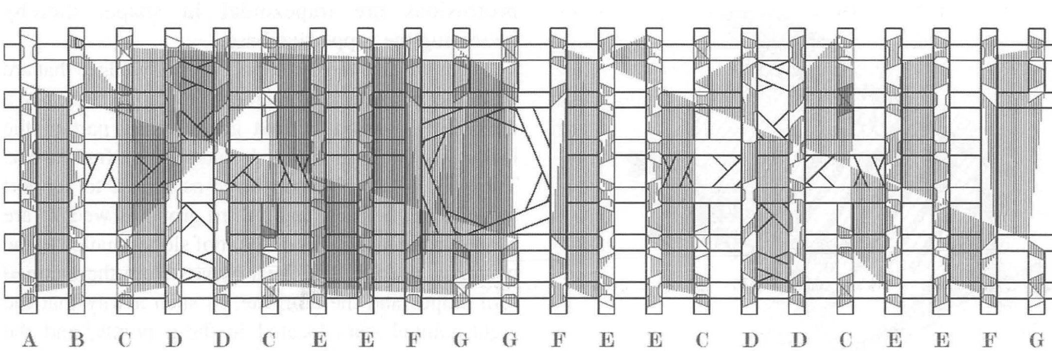

Figure 8
Types of nudillos of the Mirador

The nudillos are joined to the pares by «a garganta y quijada», following the standards and rules of carpentry. «Echando cabeza de armadura» in the alfarda we make the corresponding cut in the garganta. The cornezuelos of the nudillos are of approximately one-fourth the thickness of this element.

The type of junction between the longitudinal nudillos and peinazos is unknown, although we would dare to say that the most probable resolution is the one that traditionally has been used to unite peinazos and nudillos; that is, the joining «a romo y agudo» with the wooden nail, or espiga.

Next we will analyse the *peinazos*, which usually are located and joined perpendicular to the nudillos, thereby contributing in an important way to the stability of the almizate. he peinazos and the nudillos together transmit the pressure to the alfardas.

There are two fundamental groups of peinazos in the almizate. On the one hand, those that are situated in an orthogonal way to the nudillos (types a, b, c, d, e, f and g), and those that are placed in an oblique way, which contribute to giving a greater stability to the entire set, while at the same time serve as a base for the layout of the design of the lacework ties (types i, j, and k).

In figure 9 all the peinazos that make up the almizate are determined.

We have mentioned basically two groups of peinazos, and of these we only know for surre about

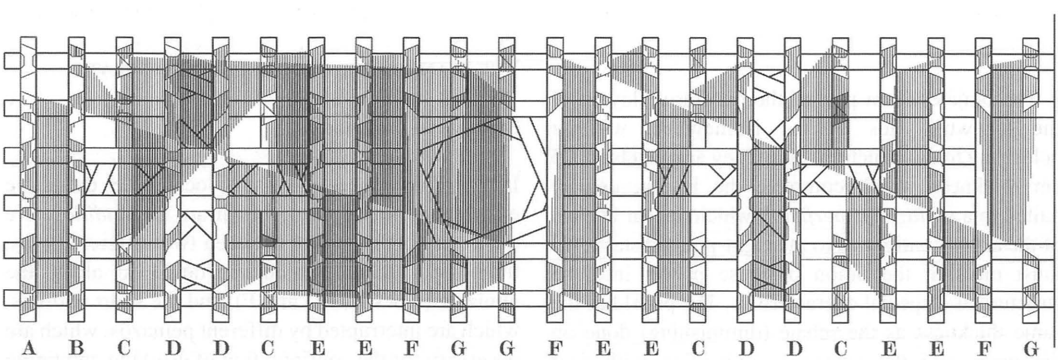

Figure 9
Types of peinazos that form part of the almizate

the unions of the types «i» and «j» to the nudillos. These materialise by means of clavazón to the resistent structure. The rest of the unions are done through different types of junctions, of which none has been accurately confirmed.

The types «a, b, c» and «f» are almost certain to be the junction of «a romo y agudo con espiga», whereas the types «d» and «h» would have this type of union at one end, while the opposite would have a «caja y espiga», with a boxed area made in the panel of the nudillo.

The junctions between the type «k» peinazos and the masters (which will be either peinazos or nudillos, depending on the position that they occupy in the almizate), have not been discovered, neither do we dare to suggest a possible form of this union. The oblique placement of the wooden pieces and the great number of elements that all come together in the same area make any supposition about the junction vain. In any case, we can affirm that for the union of the main elements that make up this pair of peinazos, a boxed area would have been hollowed out in the panel of the peinazo or nudillo and in it would have been inserted some kind of wooden peg (espiga), that will have to remain unspecified since it is hidden inside the boxed area. This peg could adopt any of many forms within the boxed area, so we will leave this matter open to the further developments of this investigation.

Finally remains to be discussed the solution adopted in the union between the peinazos. Except for the union of one of the components of the pair of peinazos of type «k» that rests on the resistent masters to open the mocárabe cube, by means of chiseling the peinazo, the rest make their unions by means of clavazón.

In figure 10 all the structural elements of the almizate are shown and the respective unions between them. Except for the junctions that are nailed and those of the entire element with the pares, the rest are all theoretical, due to the impossibility of confirming these facts, save disassembling the structure. The junctions between the peinazos that form the central octogon and the masters are the only ones that have not been defined.

Non-structural elements

It is logical that in the almizate the same elements that made up the inclined roof slopes are repeated, so we will not go into this part more profoundly than we need to. Again three types of wedges appear, and the different types of tablazones, of trasdós, of relleno, and superficial. In this case we will indicate the exact situation of each of the taujeles that make up part of the almizate in relation to the general structure.

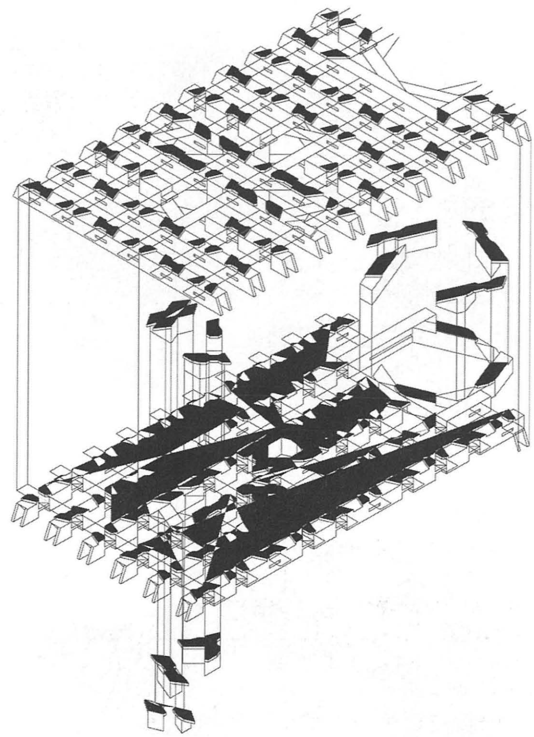

Figure 10
The structural composition of the almizate and the relation between each of its elements to the others

The same procedure is used to make the taujeles as to make the tie is the nudillos and peinazos, with the difference that in this case the little panels are cut to 1,5 cm thickness and in the former , diminishment of the same measurement has been made where part of these pieces have to fit in.

To make the different cuts, a series of set-squares is needed, used as the only plotting instrument by the carpenters «de lo blanco». These set-squares are defined by the wheel of ties or crossover lacing that they are going to plot as the design of decoration for the framework. In this case, the wheels are of eight,

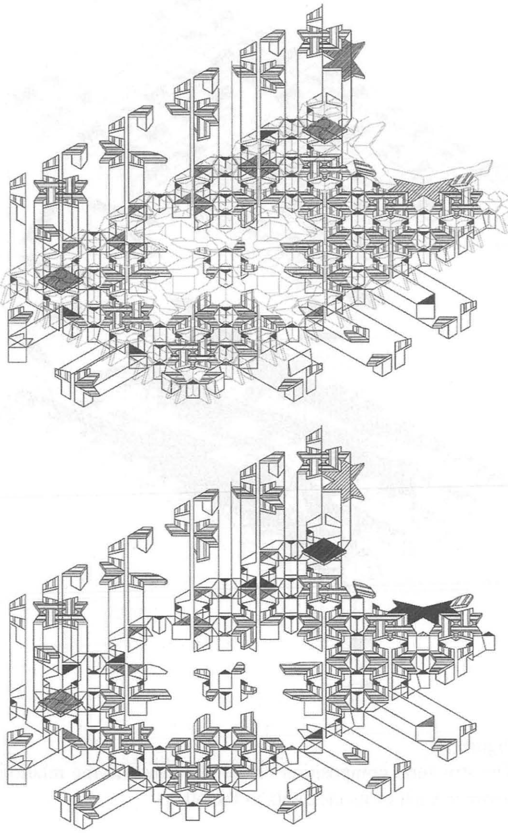

Figure 11
Non-structural elements of the almizate. The first figure shows all of the elements in relation to the rest of the structure, while in the lower part each has been isolated for a better identification

so the corresponding set-squares are therefore of eight (nominally of the tie), of a square (nominally of the theoretical wheel of a half of the arms) and of blanquillo (ataperfiles).

And finally we must make reference to the three *mocárabe* cubes that alternate with the rest of the decoration of the almizate.

Constructive development:

The first step that has to be taken is to place on top of the peinazos that make up the structural part of the octogon a panel that is in the same shape and size as the cube. The unions between the different toothing would be hidden by the peinazos of the almizate; however,it is logical to think that each tooth must be nailed upon another surface than that of its own structure. These panels are known in the theory books as albernica or alberneca. The experience of having assisted at the recent restoration of the framework confirms this fact, since the whole of the ensemble works as one sole piece supported by the structure.

Next, sixteen atacias rest upon eight eighths of the octogon and between them are fitted eight half-squares open at the widest part.

Between one atacia and the next one, eight dumbaques grullillos are fit in. (All of these elements need to have their corresponding cuts in the ends so that the «live» awns are not seen: . . . » and later upon the front of the mast itself are fit in another eight dumbaques grullillos. On these, on the high edge a lomillo is left to give it grace» . . . Fray Andrés de San Miguel, who does not specify in his theoretical paper

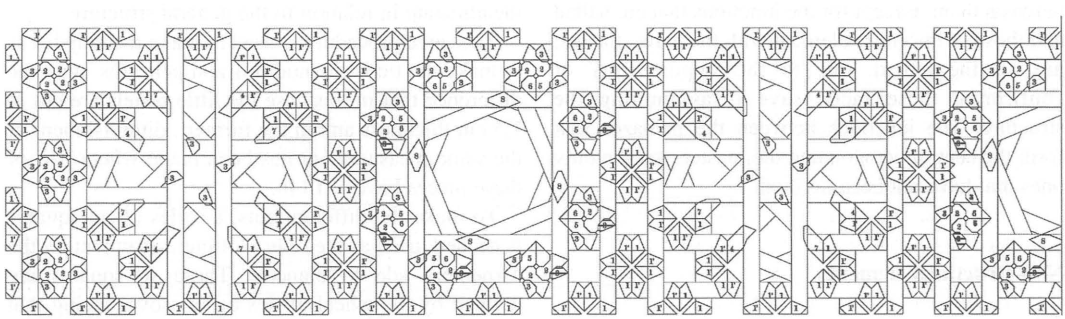

Figure 12
Types of taujeles

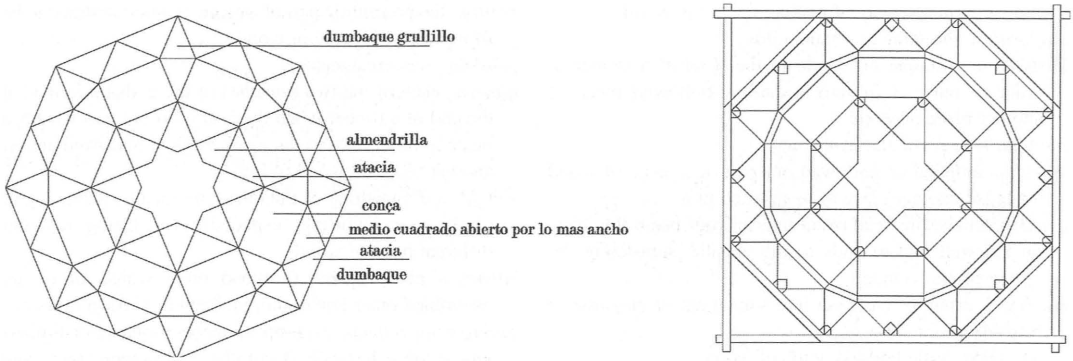

Figure 13
Geometric layout of the mocárabe cube and the final grouping, including the support structure that stays in the middle resting upon the peinazos

the precise location where the cut is to be done; supposedly these operations were sufficiently checked and done so that he could avoid giving more explanations about it).

On the half squares are placed eight conças between which another eight dumbaques are fit in, and all the pieces with their corresponding lomillos; and finally, on the conças, eight new atacias are placed and between each, to complete the composition, their corresponding almendrillas are fit in.

1) Marín Fidalgo, Ana. *El Alcázar de Sevilla bajo los Austrias*, Ediciones Guadalquivir S.L., Seville 1992
2) When the meeting between the roof slopes is resolved by two pieces, each pertaining to each of the planes of the faldones, then we are speaking about a mohamar or folded lima. This type of lima favoured the prefabrication of the faldones on the ground.
3) This consists in the creation of an orthogonal woof or plot module of the same thickness as the alfarda, calling the space occupied by the par (or the peinazo in the orthogonal direction)a cord (cuerda), and calling the space between two consecutive alfardas an aisle (calle). To follow the rule, the latter space (that is, the aisle) must be the same width as two cords. In this way the carpenter would be able to work the design of the set of precise cuts so that the wooden pieces can be placed one atop the other, producing the effect of a lacework star.
4) Peinazo: a piece of wood that is assembled with another in order to make a determined pattern, be it a door or a window, or the framework of a roofing, with or without lacing ties. Arrocaba: in the wooden framework of mohamar limas, they are the pieces that give a visual continuity to the péndolas in the aisle of limas, of the same square and rhomboid shape.

Glossary

agua. the slope of a roof
albernica. (alberneca) panel on which a toothing is nailed
alfarda. the same as «par», each of the two pieces of wood that in the roof truss of a roofing framework give the sloping to the roof
alicer. a thin strip of wood that serves as a transition between a wall and the roofing /ceiling
almendrilla. almond-shaped piece of wood used for decorative purposes
almizate. central point of the sloping of the roof in wooden roofs that are decoratively made with timbers and planks or beams that are visible
apeinazado. similar to or using the «peinazo» in construction
arrocabe. timbers situated at the top of the walls of a building to unite them with the roofing framework that they will be supporting
«a romo y agudo». a means of uniting pieces of wood, «romo» means blunt and «agudo» sharp

atacia. a particular way of making the cut in wood
ataperfiles. the same as «blanquillo»
barbilla. an oblique cut made in the front of a timber in order to make it fit into a shallow hollowed place of another piece of wood
blanquillo. a particular acute angle
cajeado. emptied or hollowed out part of a piece of wood made for another piece to be inserted in it
canes. corbels, the head of an interior roof beam that rests on the wall and extends to the outside, supporting the crown of the cornice
clavazón. group of nails put into something or prepared to be used in something
cornezuelos. little hornlike parts of wood
dumabaques grullillos. a particular type of wooden pieces used in the old roofing frameworks
«echando cabeza de armadura». when making the roofing framework, making the angle that forms the inclination or sloping of it
espiga. a type of cut imitating lines at the end of a piece of wood made so that it will be assembled and fitted with others similar, in a peg-like way
estribo. abutment; solid piece of wood made to support the weight of the roof in those parts of the wall where there would otherwise not be any support, such as the tops of doors, windows, or arches
faldón. triangular sloping of a wall
garganta. the narrowest and thinnest part of columns, balusters and other similar pieces
hilera. timber upon which the «pares» of the framework sit and which make up its ridge
lima. piece of wood located in the dihedral angle that is made by the two inclined planes of a roof, and upon which the two shorter «pares» of the framework rest
lima mohamar. a style of roofing that has two «limas» for each «faldón» or triangular slope
lomillo. decorative edging or back part or something that protrudes
mangueta. the same as a «par», except that this sits on the «lima»
media viga. a beam of 4.2 – 4.5 metres (old term)
medio ponton. a beam of 3.60 metres
mocárabe. a decorative work made of the geomatric combination of matched prisms, and that has the bottom edge cut into a concave surface
nudillo. a piece of wood that forms part of the «almizate» or central part of the roofing framework, and which joinsthe «pares»
paño. inclined surface
par. each of the two beams that in the top of the roofing framework give the inclination to the roof
patilla. the protruding part of a piece of wood destined to be fit into another piece of wood
péndola. see «mangueta»
quijera. each of the two branches of the u-shape formed at the end of a timber when the hollowed-out part has been made in order to have the «garganta» or narrowed part of another piece od wood fit into it
rebaje. a diminishing of thickness, by means of shaving or sanding or whittling, especially for fitting together different parts of wood
solera. a planed piece of wood upon which sit or are assembled other horizontal, inclined or vertical timbers
tablazón de relleno. a group of wooden planks or timbers whose job is basically decorative and to hide other more unsightly «inner workings»
tablazón superficial. panels that are placed in each of the aisles next to the eight-pointed stars
tablazón trasdós. part of the final covering of the framework, these boards are used to give a pictorial background. They are nailed to the resistent parts of the framework, the «pares» and «limas» and cover practically all of it
taujel. a slat; a long thin narrow piece of wood
terciada. a beam of 5.32 – 5.70 metres
testeros. the shorter, more secondary triangular parts of the inclination or slope of the roof
tirante. brace that can be either wooden or metallic (in this framework they are wooden)
tocadura. moulding, part of the of the «arrocabe»
viga de acarro. beam of 7.5 metres
zancar. to fix or stabilize by wedging pieces of wood or other material between two movable parts, so that they do not move any more

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Building practices of the post-war reconstruction period in Italy: Housing by Mario Ridolfi at the INA Casa Tiburtino neighbourhood in Rome (1950–54)

Rinaldo Capomolla
Rosalia Vittorini

> Ma ecco che un giorno cominciarono a impiastrare di palazzi tutto lì intorno, sulla Tiburtina, poco più su del Forte: era un'impresa dell'INA Case, e le case cominciarono a spuntare, sui prati, sui montarozzi. Avevano forme strane, coi tetti a punta, terrazzette, abbaini, finestrelle rotonde e ovali: la gente cominciava a chiamare quei caseggiati Alice nel paese delle meraviglie, Villaggio fatato, o Gerusalemme: e tutti ci ridevano . . . (Pasolini 1959, 184)

The Tiburtino neighbourhood, built by Piano INA Casa

In the early 1950s, during the difficult post-war years, a vast public housing program was begun in Italy, called *Piano INA Casa*. Construction of one of the first housing projects began in October, 1950, in the eastern outskirts of Rome, an area of approximately 9 hectares along the *Tiburtina* consular road, 7 km from the centre of the city. Approximately 770 homes were planned for 4000 inhabitants. The design group was guided by two masters of the «*scuola romana*», Ludovico Quaroni for urban planning and Mario Ridolfi for the architecture, assisted by young architects and architecture students.[1] The neighbourhood was considered a ««manifesto» of both architectural neo-realism and of the ideology of *Ina-Casa* during that first seven-year period» (Tafuri 1982, 23) and in fact the architects interpreted the building model elaborated for the *Piano Ina-Casa* almost literally.

The decision to respect and exalt the morphological characteristics of the area, adopting an architectural language which took its cues from the vernacular architecture, made for an articulated and varied urban environment. The buildings —in the form of low towers, row houses, and blocks of flats— are arranged in an irregular scheme. Together with a rich composition of roads, pedestrian pathways, terraces, galleries, green areas, vegetable gardens and piazzas with shops, they create a familiar, domestic atmosphere, reminiscent of a rural village. The result sparked the curiosity of many, even offering inspiration to Pier Paolo Pasolini, an extraordinary observer of the social fabric of the post-war period, who set one of his most famous novels in the neighbourhood. Particularly effective was his description of the roads which entered «in curva in mezzo alle case rosa, rosse, gialle tutte sbilenche esse pure, con mucchi di balconi e abbaini e sfilate di parapetti» (Pasolini 1959, 191), of «botteghe . . . ammassate in una specie di bazar a un piano al centro della borgata», and «case una addossata all'altra, a scalinata, in modo che il primo piano della seconda era all'altezza del secondo piano della prima, e così avanti: davanti alle facciatine colorate, c'erano tante scale esterne che le univano, con dei pianerottoli che facevano come da terrazzine alle porte di ingresso, tutti sbarre e inferriate» (Pasolini 1959, 318). Ridolfi interpreted quite literally the indications of the *Piano*, which suggested «an alternating play of high and low, continuous and interrupted, long and short walls,

Figure 1
M. Ridolfi housing blocks at the INA Casa Tiburtino neighbourhood, Rome

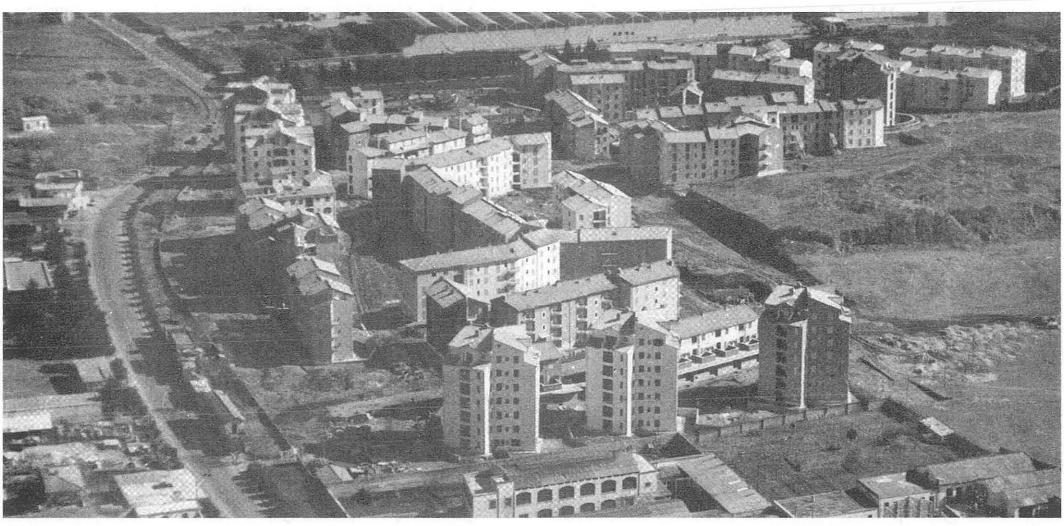

Figure 2
Aerial view of the neighbourhood (1957)

terraces, overhangs and negative spaces (windows and loggias), arranged in the facades or in the views from entry points or from the main windows of the apartments».[2]

In the overall context of the entire housing programme, the Tiburtino became a sort of pilot-neighbourhood, useful for testing both construction operations and the efficiency and extendibility of the urban, architectural and building model. More than with any other contemporary intervention, an assortment of forms and construction practices were developed which were to become the everyday armaments of the INA Casa building programme, the most immediately recognizable.

Though considered to be a «central episode of the reconstruction period in Rome» (Poretti 2002, 10) and one of the most significant expressions of architectural neo-realism, the neighbourhood provoked controversial reactions for the radical, almost anti-historic, positions it took. The designers themselves were the first to judge it, on the one hand, «out of its time», since neo-realism, which had found its most relevant expression in cinema, was already out of fashion, and, on the other hand, «out of time», because it proposed a collective living style and a romanticised, idealised identity of the rural worker to a class of blue-collar and office employees who for the most part aspired to leave that world and become a part of the emerging middle class.

Promoted by the minister of labour Amintore Fanfani, the purpose of the *Piano INA Casa* was to solve unemployment problems by hiring as much of the labour force as possible, including unspecialised workers, in the construction of housing for the working class. The idea was to finance the construction of entire neighbourhoods for the less privileged classes and for state employees, through contributions from employers, employed workers and the State. Inspired by principles of Catholic social solidarity, the law was applied through the «*Gestione INA Casa*», an agile body which directed the planning and coordination of the entire operation through a «Comitato di attuazione», an implementation committee, headed by the engineer Filippo Guala, and a «Consiglio direttivo», a managing committee, guided by the architect Arnaldo Foschini, head of the School of Architecture at the University of Rome and a leading figure of academic culture. With the aim of avoiding costly centralised bureaucratic structures, the *Gestione* entrusted design to independent professionals and assigned the working documents and construction to authorities already in operation both nationally and locally. These included the Istituto Nazionale per le Case per gli Impiegati dello Stato (National Institute for State Employee Housing), the Istituto Nazionale per la Previdenza Sociale (National Institute of Social Insurance), the Istituto Nazionale Assistenza Infortuni sul Lavoro (National Employment Accident Insurance Institute), and various Ministries, and on the local level, the Istituti provinciali per le Case Popolari (Provincial Public Housing Authorities), local administrations and building cooperatives.

The plan was part of a precise political and economic programme which considered the building sector to be the «driving force» for a general recovery of the country, a sort of «reservoir» of labour from which to draw, as necessary, during the various phases of the hoped-for industrial development. Thus it was necessary to maintain the building sector at a craftsmanship level: considering the industrial situation of the country, this would facilitate the small and medium-size industries distributed across Italy, all at a low level of mechanisation.

In order to keep building costs under control, the *Gestione* demanded well-defined design work, and maintained full control of all construction phases by means of constant monitoring at the site. Thus, updating and constant revision of design and building regulations were required.

Each and every design and building phase was carefully monitored by the Gestione through a decentralised organisation which entrusted every operation to the public body contracting the project, from selection of the architects, to tendering procedures and project management, to final inspection and assignment of the dwelling units. The selected designers were inserted in special lists. One third of the entire class of design professionals, often successful architects and engineers teamed up with new graduates, were given work opportunities through the programme.

In 1949–50, the *Gestione* published two booklets whose aim was to unify the buildings, both economically and from the point of view of architecture and planning. The booklets contained suggestions, regulations and examples, as well as typical projects, both in terms of architecture and

Figure 3
The «linear» blocks and the «anfora» shaped building on via Tiburtina (1957)

construction —with typological models and rules concerning techniques and materials— and in terms of planning. For the latter, the neighbourhoods inspired by the language of the «New Empiricism» elaborated in the Scandinavian countries were indicated. The architects were asked to avoid standardised layouts and types in order to «give the inhabitants of the new urban centres the impression

Figure 4
One of the shop on via D. Angeli (1957)

Figure 5
The balcony flats on via L. Cesana (1957)

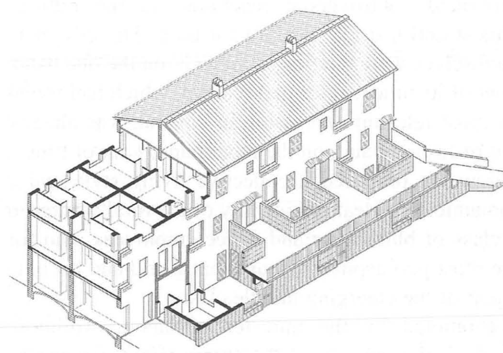

Figure 6
The balcony flats on via D. Angeli. Axonometric view, R. Vittorini

Figure 7
The star shaped towers and the balcony flats on via D. Angeli (1957)

that their home is a spontaneous, authentic and permanent part of the local area». In terms of design programming, the Piano's building programme was defined as «psychological building», intended, that is, to provide the best possible environmental conditions for the daily lives of the workers. The architects were explicitly instructed to provide high-quality construction, to give the dwellings an «air» of dignity, and to ensure a comfortable environment. This is the manner in which the difficult and compelling theme of public housing was approached on a large scale, in line with the conservative spirit of the *Piano* which called for an programme rooted in the local area, with studies of the local architectural characteristics, climate and materials, taking into account «topographic characteristics, local resources, green areas, and views» (De' Cocci [1957], 96). The designers were required to carry out «in-depth studies of all technical and architectural details, in terms of layout and construction efficiency (economy of space, materials and time); from a human point of view (design of the units based on the well-being of the family); and from an aesthetic point of view (general architectural level of the building project)».[3] The adoption of conventional construction methods was advised, in order to preserve the craftsmanship skills of the workers, as explicitly described in the strategy of the plan. The designers were pressed to «remind contractors of their obligation to perform all building processes correctly by means of carefully drawn construction details and proper headings in the specifications, included as part of the construction documents».[4]

During the two seven-year periods of application of the Fanfani Law, from 1949 to 1963, approximately 350,000 housing units were built. Entire autonomous neighbourhoods, often abounding with services, public spaces and parks, sprung up across the entire country, from large and medium-size cities to small mountain villages and the towns of the large and the smaller islands. The intense level of design and building activity, in terms of both size and quality, encouraged widespread debate throughout Italy and favoured unprecedented experimentation, which was, however, distant and different from that of the other European countries, where housing programmes were directly tied to the modernization of the building sector, through the study and application of advanced technologies regarding unification and prefabrication.

A MODEL OF «ADVANCED CRAFTSMANSHIP»

In the *Tiburtino* project, there was a remarkable «consonance» between the economic, political and social objectives of the *Piano* and the efforts of the designers to create a concrete architecture which could interpret the «values» and the aspirations of the lower classes.

Even now that the neighbourhood is integrated into the city and no longer a frontier of expansion, it clearly stands out from the speculative building of the surrounding suburbs. In walking its streets, one senses a «rural» atmosphere, reminiscent of certain villages in the Rome countryside. This atmosphere is owed first of all to the layout (the «stage», to use an expression by Zevi). The buildings constantly change direction in response to the topography of the site, creating appealing perspective views, while piazzas and wider places in the road, devoted to socialisation between inhabitants, flow into more intimate, domestic environments between the buildings, facilitating relations between neighbours. This rural air is also due to the «dialectal language» of the facades, where the typical characteristics of traditional masonry construction are proposed and reinterpreted.

The usual construction method was brick or stone bearing walls with bond beams, lintels and floors in reinforced concrete. Already well-established at the end of the nineteenth century, this traditional technique had revealed itself to be so perfectly adapted to the economic and productive situation of Italy that it developed and spread throughout the 1930s and '40s, during the height of economic autarchy. The preference given by *INA Casa* for this construction method, considered particularly suitable for buildings with just a few floors, were based on reasons of cost and «custom».

This was in fact the method adopted at *Tiburtino*, in accordance with the indications of the *Gestione* which suggested the use of bearing walls constructed with «the most suitable materials in terms of strength, durability, insulation, etc., and at the same time the most economical for the area in which the houses are to be built [. . .] avoiding long and contorted layouts, eliminating large opening in the structures and large open floor spaces, [. . . avoiding] balconies with large overhangs».[5] Ridolfi used the traditional masonry «*alla romana*», consisting of blocks of volcanic tufa

could reach the floor above with a half or full turn, always clockwise, until it reached the summit of the tower where originally there was a terrace.[3]

The interwall staircase is an ever present element within the nuraghi, and two building techniques can be found in the thousands of towers *«scala di camera»* and *«scala d'andito»*.

The first type begins from the main chamber and at a certain height from the ground (for example at Nuraghe Is Paras-Isili it begins at a height of 5.50 metres) and passing between the walls reaches the next floor. In most of these types of nuraghi, the base chamber is centrally placed in relation to the external diameter of the building and the thickness of the walls is insufficient to support the beginning of the staircase at this height. In fact the incline of the chamber wall determines, at a certain height from the ground, the height sufficient for the passage of the staircase.

The «scala d'andito» is, on the other hand, technically more advanced and probabily more recent, because its construction presupposed a greater design ability compared to the previous one due to control of the whole elicoidal development of the interwall staircase and the subsequent positioning of the upper chambers. The staircase has, in fact, a route from ground level to the top of the tower with spaces only for the entrance to the upper chambers. The circumference of the ground floor chamber is not concentric to the external circumference of the tower which means in section, the eccentricity of the chambers with regard to a vertical axis sited at the centre of the external circumference.

Building technique

At the beginning of the construction of a complex and articulated building such as a nuraghe there is a plan, a theoretical and methodological formulation, which permits control of its completion. Building is not based on improvisation or spontaneous invention, but must be the result of the fusion of construction and project. It would be wrong, in fact, to consider the act of building and planning as casual. Planning and construction run side by side and are the result of the consolidation of acquired experience and building practice. A nuraghe was a construction in which structure and functionality were integrated, and there are no superfluous or secondary elements, every single element takes part in the stability of the whole. Studies so far undertaken into the building techniques of the nuraghi have been interested in the methods of transport and positioning of the enormous blocks of stone needed for their construction.

The realisation of a work such as a nuraghe was a rather complex operation, requiring a notable use of resources and human energy as well as a functional organisation of the building site. Given the number of towers built it is reasonable to think that the building technique was the patrimony of the Sardinian people. They probably had on hand a clear building project, knew, possibly empirically, the caracteristics of their materials, and above all knew exactly, given their experience, which were the critical points of the sructure. In fact we find throughout the territory the same structural scheme, at least in its essentials.

We can imagine the coordinated work of a number of teams who undertook the choice, cutting and laying of the blocks. Machinery was used for raising the blocks. In fact a deliberately carved stone was found near Nuraghe S. Cristina di Paulilatino, which was probably used as a counterweight on the building site.[4]

The first phase of the construction was establishing the building plan and therefore the organisation of the internal space of the tower and architectural choices such as the type of staircase, which as we have seen, determines and defines the details of the upper floors. As the construction rose, level by level,[5] spaces were defined, full and open, which defined the final structure. The critical points were the open spaces such as openings, niches and especially the route of the intermural staircases. In the upper levels the external walls reduce in thickness (from medium at the base of 4 metres to a medium of 1.5 metres in the upper layers). This was due to both to the more regularly cut stones used at this level to the more regulary levels of the layers, and to the inclination of the external wall.

The tholos, the system of covering the nuragic chamber, is created in dry stone concentric rings laid on above the other horizontallt. Every successive ring is positioned inward with respect to the lower one in order to produce a vertical section with a curved profile. As the building rises the blocks are smaller and the roof ends with the smallest blocks which are them selves closed with a cap stone. The resultant self supporting structure is capable of remaining standing

during construction without the help of any system of centring.

The tholos is a revolving structure which has in fact both horizontal circular section and vertical symetrical section in relation to an axis which coicides with the perpendicular at ground level which pass through the apex. The volume determined by this structure is obtained by the rotation around the axis of symetry of the profile of the intrados. A structure geometrically defined in this way cannot be realised without the help of a building method capable of controlling its horizontal development, vertically and radially.

The aspect of the building technique on which this study concentrates is the discovery of a method, simple and efficient, for the determination and control in the building stage, of the archway soffit (intrados) of the tholos.

A building method to create an overhanging roof with the form of a nuragic roof has been proposed by two English scholars W. G. Cavanagh and R. R. Laxton[6] who believe, after having studied 15 nuragic tholoi, that simple instruments such as treetrunks, sticks and ropes would suffice to check the curvature of the chamber as construction goes ahead. The method[7] (Fig. 1) which they propose consists of siting a trunk of height equal to H units (one unit equals half of the layer height) at the centre of the chamber in a vertical position, place a wooden beam perpendicular to the trunck on the last layer built, tie a rope to the top of the trunk and move it as far as the upper edge inside the last complete layer.

Then one runs horizontally on the wooden beam a stick of two units of height, marked at the middle, until the rope passes this point, the next level must be positioned until it touches the stick. The same manouvre is repeated until the roof is closed.

This hypothesis, though based on the use of simple tools, is from the constructional point of view complicated and restrictive. In fact the method as proposed is applicable only to the tholoi which have a constant height of layers. Also, if the technique were used to construct a nuragic tholos with layers of a decreasing height from below to above, it would be necessary to have a different stick for each height of layer, making the method even more complex.

The present work, as an alternative to that of Cavanagh and Laxton for the control of the curvature of the tholos is based on a building technique which

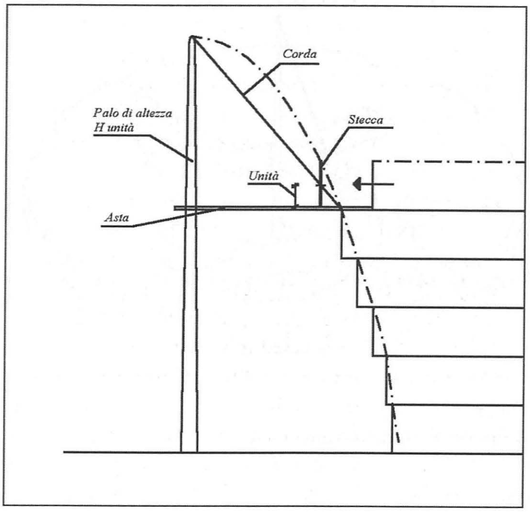

Figure1
Method proposed by Cavanagh and Laxton for the determination of the curvature of the tholos

uses simple tools such as trunks, ropes and a plumbline.

The first construction phase of a nuraghe consists of establishing in plan the internal spacial organisation of the first level. Thus the external dimension of the tower, and the position of the chamber and its diameter are established. Having completed the first layer of stones (Fig. 2) a trunk of height H slightly inferior to the real height of the roof is sited in the centre of the chamber in a vertical position.

At the top of the pole is fixed a rope of a length egual to the distance from the point at the top of the pole to the upper internal edge of the first layer, plus the height of the layer itself.

At the bottom end of the rope a weight is attached producing a plumbline.

The overhang of the levels is determined thus: the rope is stretched from the top of the pole as far as the upper internal edge of the stone already laid, then the stone is moved until the weight at the end of the rope touches the ground. The same operation can be undertaken radially as many times as is necessary to position the various stones of the same layer. The process then continues layer by layer (Fig. 3).

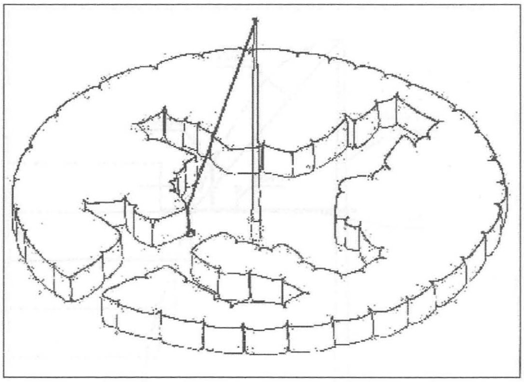

Figure 2
Description of the building method

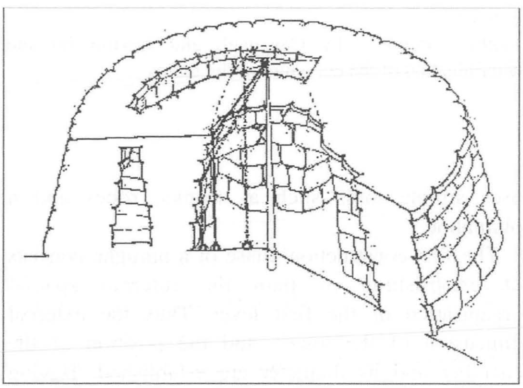

Figure 3

With this method it is possible to check the circular nature of the horizontal layers and to establish that as the tholos is built, the growth of the whole building is controlled horizontally. In this way the part already built becomes the base upon which the builders could work and control the whole internal spacial organisation which otherwise would be too difficult to manage.

THE GEOMETRIC CHARACTERISTICS OF THE CURVE

The curve obtained with this building method is a parabola and the volume of the tholos is a paraboloid.

To better understand the chracteristics of the construction and the hypothesis proposed it is better to observe the geometrc properties of this conic.

The parabola is the geometric locus of the points on the plane equidistant from a fixed point called the Focus and from a straight line called the Directrix. It is also possible to define the parabola as a Limit Ellipse, and therefore as the geometric locus of the points on the plane for which the sum of the distances from two fixed points called Focus which is constant and belongs to the axis of symmetry and of which one to infinity. (Fig. 4)

To verify whether the curve obtained by sectioning the nuragic tholos vertically through its axis is a parabola, the data obtained by Cavanagh and Laxton on a sample of pseudovolts were used.

Among the data published the following sections of nuragic tholoi were closen, Is Paras-Isili, Palmavera-Alghero, S. Sabina-Silanus, the first and second chambers of S. Antine-Torralba and Orolio-Silanus. For each tholos the two scholars provide the coordinates of the points found during the measurement, referring to a Cartesian system beginning at the top of the tholos.

Having considered the generic function of the parabola $y = -ax^2$ (the negative sign indicates the concavity of the curve downwards) and notes x y coordinates of the points in the tholos, the coefficient to determine the function of every single section remains unknown.

Assigning to x and respectively the values of the radius (regarding the axis of the tholos) and of the depht (regarding the apex) of a point chosen, based on the most probable parabolic alignment of the points, the coefficient of the parabola can be found.

For a generic point P we therefore have x_p and y_p and from the equation

$y_p = -ax_p^2$ we obtain the coefficient $a = -\dfrac{y_P}{x_P^2}$.

At this point it is possible to obtain the function of the parabola with the data of Cavanagh and Laxton. Such comparisons (Figs. 6, 7, 8, 9, 10, 11, 12) and in particular the one relative to the figures of Nuraghe S. Sabina-Silanus would confirm the validity of my hypothesis.[8]

The tholos is therefore a structure with a geometry which is also a building rule.

In order to relate the equation of the parabola $y = -ax^2$ to the building method proposed it is necessari

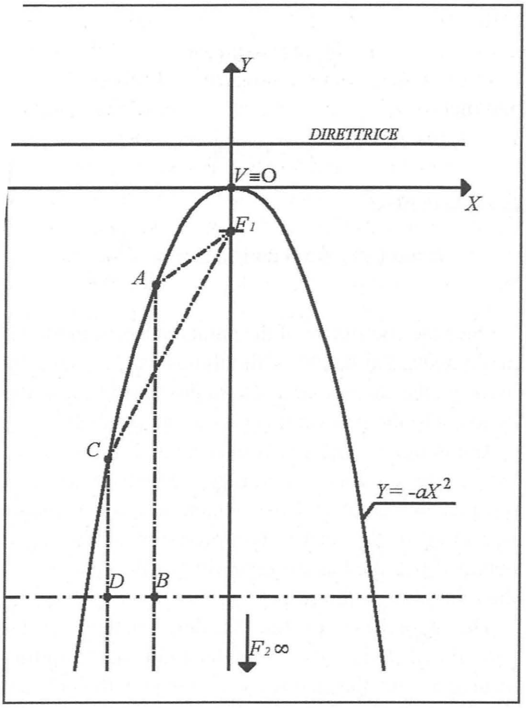

Figure 4

$$\overline{F_1A} + \overline{AF_2} = K$$
$$\overline{F_1C} + \overline{CF_2} = K$$

then

$$\overline{BF_2} = \infty = K^I$$
$$\overline{DF_2} = \infty = K^I$$

and still

$$\overline{F_1A} + \overline{AF_2} - \overline{BF_2} = K - K^I = K^{II}$$
$$\overline{F_1C} + \overline{CF_2} - \overline{DF_2} = K - K^I = K^{II}$$

so

$$\overline{F_1A} + \overline{AB}\, K^{II}$$
$$\overline{F_1C} + \overline{CD}\, K^{II}$$

to specify the dimensional parameters which determine the coefficient. This defines the form of the parabola, as $a = \frac{1}{2p}$ with p equal to the distance between the Focus and the Diretrix; in the proposed method the top of the pole represents the Focus, and the parameter p of the parabola the difference between the lenght of the rope and the height of the pole. To determine the Focus geometrically it is sufficient to prolung a tangent to the curve until it intersects the axis of the x and trace a perpendicular from this point to the tangent line until it intersects the axis of y. This defines the height of the pole which is equal to the height of the chamber subtracted from the distance Focus-Vertex.

The length of the rope is equal to the sum of the double distance Focus-Vertex plus the height of the pole. The choise of the profile to give to the chambers of the nuraghi is the result of a spacial optimization aimed at reducing the full-empty relationship.

To enclose a space with a circular base it is held more advantageous, with equal heightand diameter, a parabolic rather than a conical profile, because of the greater saving in material and the subsequent load reduction.

Furthermore, at equal height, more space vertically is obtained, and so there is the chance to have, through the use of wooden platforms, more useful space.

In fact these buildings are in a poor state of conservation. Often the tholos are without their upper levels incomplete. To hypothesis the original height could be important, therefore, in valuing the history of this architecture if we accept the hypothesis so far given as valid, we can have a means of defining the height of the tholos which are reduced to the state of ruins.a

After having undertaken an accurate survey of the remains of the tholos, the measurements are put into a Cartesian system, and you draw the curve which passes through the points which show the most probable alignment.

Definided as B and C, respectively the lowest and highest of the curve, we assume x_B and x_C as radii of the tholos with respect to B and C. Given the implicit equation of the parabola $y = -ax^2$ and the Cartesian coordinates of points B and C, it is necesary to find the value of the coefficient a to determine the equation of the parabola which passes through the points of most probable alignment.

The equation for the required parabola has a coefficient negative and does not show the constant so the conic has a vertex at its origin and concavity towards the ground.

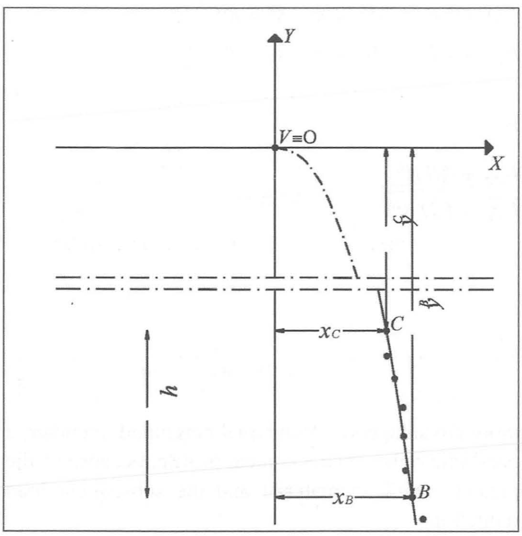

Figure 5

In this Cartesian system (Fig. 5) we note in the curve surveyed the coordinates x_B and x_C, but the ordinates are obviously unknown. Of the ordinates y_B and y_C it is however noted their difference which is equal to the distanceon the between B and C.

If

$$y_B = -ax^2 \quad \text{and} \quad y_C = -ax_C^2$$

then

$$y_B - y_C = -ax_B^2 - (-ax_C^2);$$

[1]

$$y_B - y_C = a\,(x_B^2)$$

and that

$$y_B - y_C = h,$$

la [1] becomes:

$$h = a\,(-x_B^2 + x_B^2) \text{ and so } a = \frac{h}{(x_C^2 - x_B^2)}.$$

Once the coefficient of the parabola has been found, the hypothetical height of the tholos can be found by inserting the values of a and x, radius of the base of the tholos, into the generic function of the parabola.

Theoretically this method is valid for the tholos which are no longer complete, which preserve a residual height of at least 3 metres and a noteable beginning to the curve. The precision of the result obtained is linked to the regularity of the section and the reliability of the survey.

The hypothesis of the building method so far formulated helps us to understand the building technique and the geometry of nuragic tholoi,[9] and therefore allowus to describe theirspacial configuration and structure.

This is the starting point towards the definition of a geometric model before an structural analysis of the tholoi, a subject at the moment under study.

The necessity to investigate and understand the dynamic of the whole construction is dictated by the conviction that in order to preserve the historical testimony an adeguate technical-structural knowledge is necessary.

At the moment the nuraghi (but this applies also to other nuragic construction such as a sacred Wells and Giants'tombs) are in a very poor state, caused by the passage of time and because of attemps at consolidation of an irreversible nature used until now. Unfortunately the use of modern materials such as cement, metal bars and resins introduced via perforation cause permanent damage to the monument.

The work so far undertaken has not been done to resolve forever the problems inherent in the construction technique originally used, but should be seen as a first step towards an understanding of the structure, in order to make interventions possible which are less destructive which will allow a better preservation of these witnesses of our past.

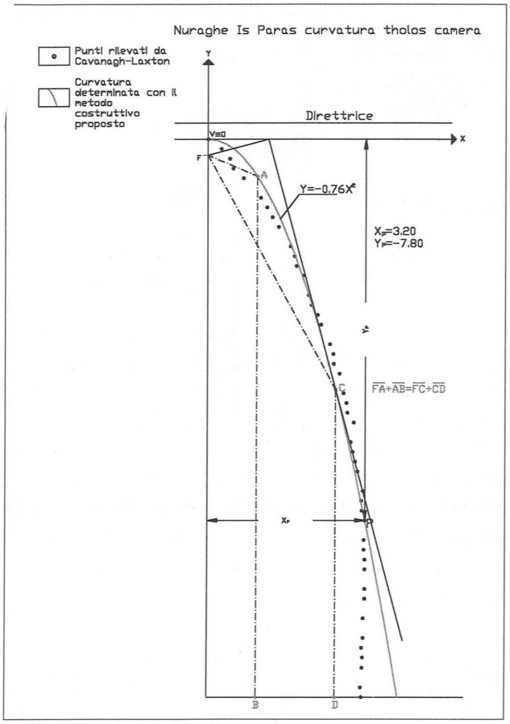

Figure 6
Curvature of the tholos in the Nuraghe Is Paras-Isili

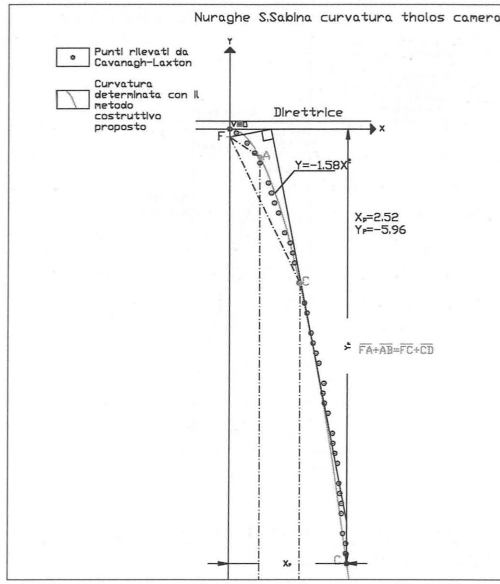

Figura 8
Curvature of the tholos in the Nuraghe S. Sabina-Silanus

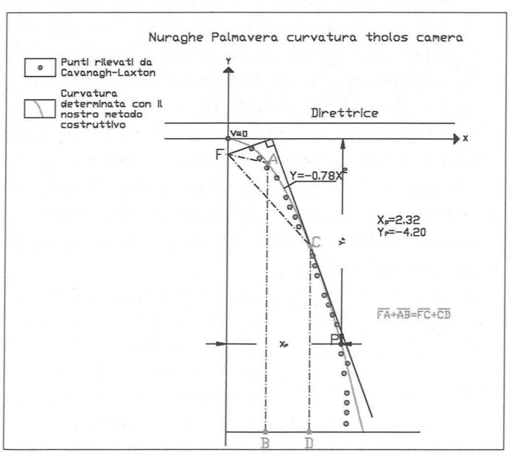

Figura 7
Curvature of the tholos in the Nuraghe Palmavera-Alghero

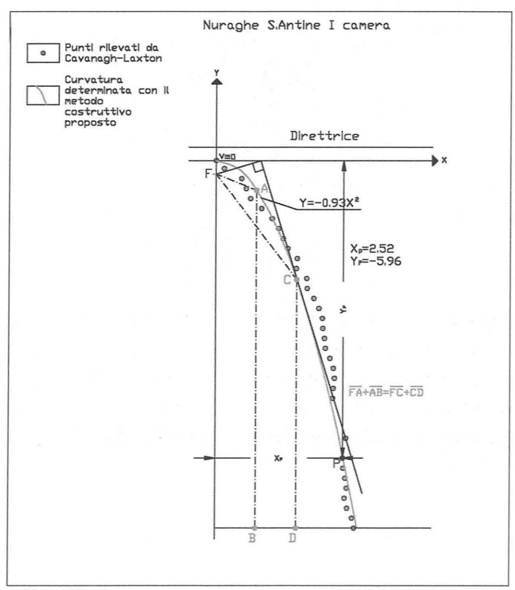

Figure 9
Curvature of the tholos in the first chamber of the Nuraghe S. Antine-Torralba

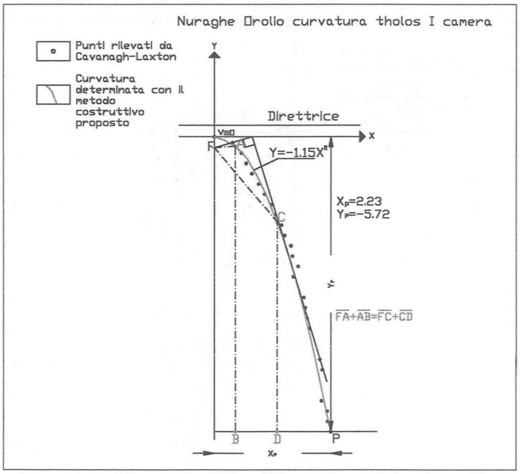

Figure 10
Curvature of the tholos in the second chamber of the Nuraghe S. Antine-Torralba

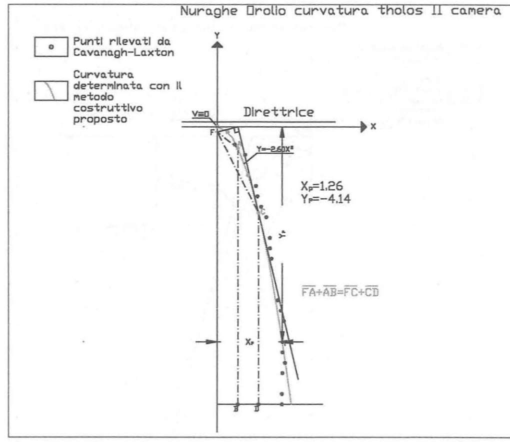

Figure 12
Curvature of the tholos in the second chamber of the Nuraghe Orolio-Silanus

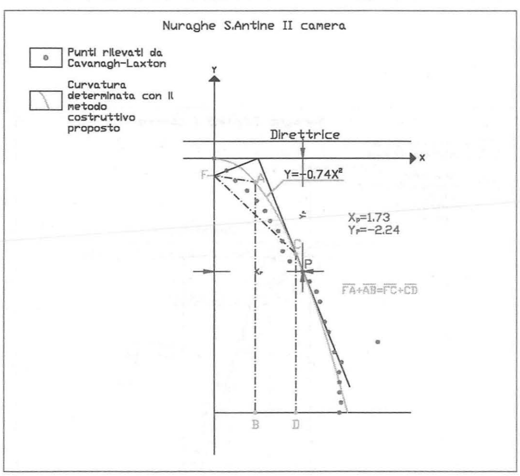

Figure 11
Curvature of the tholos in the first chamber of the Nuraghe Orolio-Silanus

Notes

* This work is part of my graduating thesis written and presented at the Facoltà di Architettura di Firenze (a.a. 1999–2000), together with the collegue and friend Arch. Giuseppe Pulina to whom goes my gratitude.
Furthermore I would like to thank the Fondazione del Banco di Sardegna that made possibile my research work «Torri nuragiche: caratteristiche tecnico-strutturali»; Prof. Luciano Barbi of the fac. Architettura di Firenze, Prof. Barbara De Nicolo of the Fac. Ingegneria di Cagliari for the precious suggestions and the constant support, and last but not least David Bollart for the care had in translating this work.

1. The expression «Nuraghe» comes from the word «nurra» which in the Sardinian dialect means «heap» or «cavità» and so a hollow construction. Nuraghi are mentioned in classical literature by the Greeks as *dedalei* and *tholos* and by the Romans as *castra* (castles) and *spelonche.*
2. If, in fact,we observe the geolithological map of Sardinia compared to the density of nuraghi it is evident that, in the flat lands and alluvial plin these archetectural outcrops become rarer until they disappear all together. The explanation can be found in the fact that building became more difficult in those areas in which the most suitable lithoid material was ot immediately available on the surface.
3. Originally a nuraghe had an over hanging parapet built

into its highest level, resting on alternative corbels and stone blocks and forming an integral part of the underlying wall. The discovery in the piles of stone caused by collapse, found at the base of several towers cut in corbel shape, and the finding of models of nuraghi showing this type of roof would confirm this type of overhang used to sustain an open terrace.

4. The use of wood, not only in the building stage but also within the finished nuraghi, has been found in some nuraghi in the form of internal-wall holes for the positioning of wooden roof-beams. Nuraghe Oes-Giave, a nuraghe with two external towers with a «scala d'adito» i this respect is of particular interest. Its interior is an open cylinder, originally covered by a single tholos. This environment was probably divided vertically by wooden garrets, resting on continuous horizontal stone ledges, one for each floor, joined by an interwall staircase of an elicoidal shape.this is a very refined solution of spacial use because the chambers preserve more or less the same size and do not diminish in diameter with height. Energy is also saved by avoiding the construction of a tholos for every chamber.
5. The Nuraghe Ruggiu-Chiaramonti (Fig.), a single tower with tholos and «scala d'andito» has inits interior at level of the passage entrance, stone slabs between the outer walls of the tower and the inner walls of the tholos, sunk, therefore, into the two parameters of the nuraghe. This particular feature, which can be found also in other cases, bears witness to the fact that the realisation of the tholos and the external walls of the tower took place at the same time and on horizontal planes, thus utilizing as a building base those layers already constructed.
6. W. G. Cavanagh and R. R. Laxton «An investigation into the construction of Sardinian Nuraghi», 1987.
7. The building method adopted for 15 nuragic tholoi is described as follow in their pubblication: *«Our analysis is based on carefully measured sections of the domes. A rapid method on surveying was devised using lasers. In the final season this involved mounting a sell levelling rotating laser at the centre of the vault, so that a diametric vertical section could be defined. A theodolite (a Kern DKM2), with a laser eyepiece mounted, was set up as far from the section as space would allow. The precise location of the theodolite in relation to the given section was calculated. The two laser spots were then aimed to concide at a series of points round the sections as defined by the rotating laser. The position of each point could be calculated from the vertical and horizontal readings on the theodolite and the known perpendicular distance of the theodolite from the line of the section. This method is sensitive to knowing the precise location of the theodolite, but our measurement were found to be accurated to ± 1%»*. W. G. Cavanagh and R. R. Laxton «An investigation into the costruction of Sardinian Nuraghi», 1987.
8. The discrepancy found in the alignment between the curve determined according to my hypothesis and the points discovered by Cavanagh and Laxton, are due to the summary working of the visible faces of the stone blocks, as the profile of the intrados is not regular. Not only, but the survey undertaken by measuring the rays at a constant but too short distance do not take into consideration the horizontal flights of the layers produces the error.
9. The principle on which the construction of nuragic tholoi is based, as we have seen, is the progressive overhanging of the stone blocks. The same criteria can be found in the other types of architectural construction built by the Nuragic peolple, in the hypogeic tholoi of the holyweels, in the system of closure of the corridors of the bastion in Complex nuraghi, as also in the single corrido of the Giants' Tomb. It is also possible the hypothesise in these last two cases the use of the method used for the tholos, with the difference that, in the case of the corridors, there is a translation of the parabola along a horizontal axis and not radially as happens in the tholoi. This subject is at the moment being studied.

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The suspension bridge by iron chains on the Garigliano Real Ferdinando. An example of innovative construction technique in Naples and Italy in the Bourbon Age in 1832

Agostino Catalano

«The suspension bridges by iron chains always rise doubts on their solidity. Even without knowing the laws of the mechanics, it will be easy to realize how difficult it is to try such a venture». By this assertion, reported on a nineteenth-century publication, it is clear how the spread of these daring constructions struck the common imagery all over Europe. Therefore, we can affirm that in the history of science and technique, the construction of a bridge is one of the most interesting chapter both for the technical value of the structure and for its cultural implications. The bridge, considered one among the main constructions of the engineering, has stirred up, since the beginning of the civilization, the creativity and the technical ability of homo sapiens who probably saw in it the first incentive towards the progress, as Kubrick could have pointed out in his memorable film «2001: A Space Odyssey». The famous scene in which the bone turns into a space shuttle symbolizes the developing arch of human achievements; through this representation, as engineers, we have to consider that structure as a creation which syntheses in itself not only the high specialized nature of the technique but even the extraordinary expression of the constructive excellence which it arouses.

As a matter of fact, the suspension bridges represented the expression of a culture which, between the eighteenth and the beginning of the nineteenth century, thanks to the economic growth of the Industrial Revolution in England, saw the development of the technical thought which spread over many social fields and unburdened the twentieth century of the load of the Romanticism.

The scientific thought developed free and progressive, while Europe was involved in the struggle between liberals and supporters of the absolutist monarchies which were always been hostile to the sciences and to the economic development. This revolution, conceived as the pursuit of both a new political establishment and thought, led to the creation of the new middle twentieth century Europe; it had as consequence the end of that contrast between rationality and history which French Revolution was based on.

The consequence, quoting Benedetto Croce's words, was that «. . . the breaking of the bonds, which had hindered and kept on hindering industry and trade development, was an effect of the need to vent the creativity, the individual value and the competition, and of the need to increase the wealth which, generated by everybody or everybody belonging, was however the wealth of whole society and just for this reason it was useful».

The researches in the constructive and static techniques of suspension bridges developed in an age characterized by monumental engineering works in several fields and by the use of the iron, which gave market opportunities to the industry. Besides such advanced studies were closely connected to the scientific thought, developed by researchers in the field of civil engineering on the static laws and its related applications. Furthermore the necessity of the

bridge constructions involved an alteration of the scenery, increased by the usage of a non-natural material like the iron which took the place of the wood; it is important, however, to point out that a suspension bridge was a more suitable supplementary part of the scenery compared with the masonry bridges built since the ancient times. Nevertheless the necessity of developing the construction of the bridges complied with the man's will who wanted to achieve something concrete, which not only changed the scenery but even had a remarkable utility; in fact the idea of *scenery* is closely connected to the idea of *place*, conceived not only as a location of buildings but especially as a complex «. . . including concrete things with their material substance, shape, suitable placing and colour». Considering the engineering and the architecture to be at man's disposal for making easier his living conditions; it is clear that this new man, born of the French Revolution, naturally aimed at realizing such constructions. The importance of bridge implementations, in nineteenth century, did not merely rise from trade and military necessities, but even from the wish to satisfy the cultural thirst, born after the darkness of the reason. It was necessary, to exploit the places at their best, to come to a compromise with the *genius loci*, conceived «as the whole of the difficulties which man has to face to achieve the opportunity of dwelling»: as regards the bridges, the difficulties correspond to the geomorphic peculiarities of the land.

This introduction aims at explaining how, in that period, the cultural impulse gave rise both to researches and constructions all over Europe; examples of this cultural impulse were the several suspension bridges, planned and built particularly in England and in Russia by skilful engineers supported by sovereigns, inclined to such innovations as conscious of their benefits for the territory distribution and for the economy. Besides the bridge constructions were supported by France which, after Napoleonic conquests, had the necessity of organizing militarily the wide territory of the Empire; therefore the bridges represented the natural shortening of the distances for the transport of the troops. As a consequence of these necessities, engineers specialized in the implementation of such structures were born, inspired by Rondelet's treatises both for cultural and constructive aspects. In order to understand the cultural conditions and the technical knowledges in which the planning of the suspension bridges developed, we will describe some of them.

Among the first constructions, the English bridges are noteworthy: the Bridge on the Straits of Menai, planned in 1818 and opened to the public in January 1826, and the Conway Bridge, linking England to Ireland and built between 1822 and 1826, were both realized by Telford, and the Bridge of Hammersmith was built by Clark and Brown between 1825 and 1827. Besides it is important to mention the three suspension bridges constructed in Russia in Saint Petersburg, named *Egyptian, of the Four Lions* and *of the Four Gryphons*, built by Traitteur between 1825 and 1826.

The planning genesis of the bridge on the Straits of Menai was very complicated. The first plan, dated to 1801 by the engineer Ronnie, could not be realized because of the exorbitant price estimated at 259.140 pounds. After considering several solutions, all of them impracticable for different technical reasons, in 1810, after Ronnie's death, the project was commissioned to the engineer Telford. After discarding a first plan with big arches, he projected a single span suspension bridge made up of iron and cast iron with a 500 feet span and a 100 feet maximum height above sea level. This project was substituted with another one, characterized by two iron-cast pyramidal piers, supporting a 560 feet span. The chains had to be realized with six cables, each of them composed of 36 square section bars; a metallic bar, spirally wounded round the nucleus, contained each chain, characterized by a four inches diameter. All the chains were anchored in the masonry piers, placed to the ends of the shores.

Telford's projects were supported by load and deformation tests, carried out on iron bars by Barlow, a mathematics teacher, who estimated at 29 tons the strength value of the constructive system of the catenaries. Telford improved the experimentation, achieving a good knowledge of the behaviour even thanks to the structural calculus method formulated by Provis, his teacher, and determining a chain section of 1 square inch to which a 11 tons testing load is applied. The experimentation was carried out by a machine, invented by Provis, based on the calculus of the friction generated by the stretch of the bar under tension; the bar was hit with strokes of hammer which produced vibrations useful to calculate the bar strength even under non-axial stress.

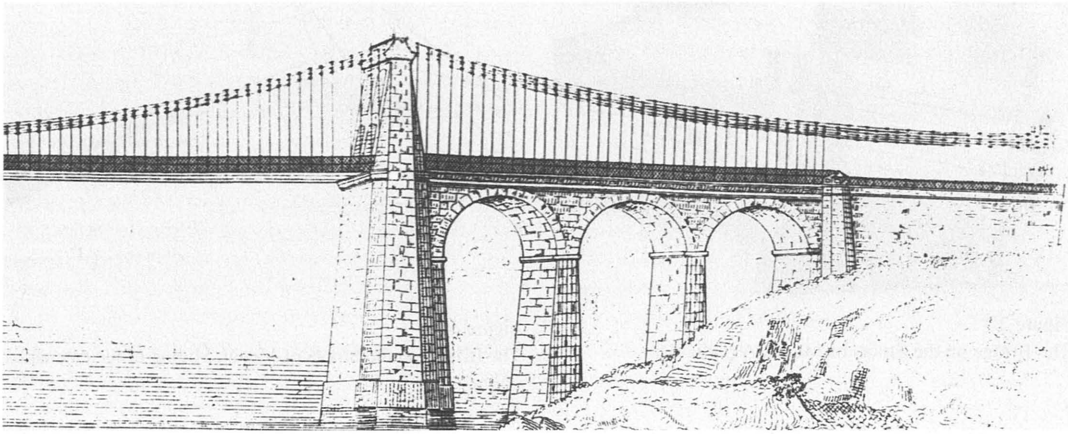

Figure 1
The Bridge on the Straits of Menai

The result was considered positive according to the elongation shown by the bar (fig. 1).

After the tests preparatory to the executive planning, the chosen solution was characterized by four and three stony arches which, placed on the shores, delimited two frustums of pyramid. In each of the two frustums of pyramid, four cast-iron plates were placed, each of them should clamp twenty chains in a vice, for a total of 80 chains, for the construction of the bridge characterized by a 28 feet wide deck, divided into two roads separated by a pedestrian crossing.

As concerns the executive planning, Telford found a successful solution to divide the loads equally on the pyramidal sections, in order to allow the minimum sliding of the chains to avoid any expansion. The problems related to the thermal expansion and to the swings generated by wind effect were solved inserting into the stony section cast-iron cushions, on which cylinders, permitting the sliding, were laid. Telford paid his attention even to the bridge corrosion, protecting the iron structures with the so-called «oil plaster». The metallic components were dipped into linen oil soon after its heating in a kiln at a very high temperature. After the bath, the metallic components were put again into the kiln, at a lower temperature for a period lasting three or four hours, in order to have a deep diffusion of the linen oil into the pores of the metal. This procedure finished by rubbing those metallic parts with oil-soaked flannel clothes.

The construction of the Conway Bridge started in April 1822 and ended in 1826; it was projected by Telford and accomplished by Provis. In order to place the structure, which had to link the English shore to the Irish one, in harmony with the scenery dominated by the ruins of an old castle, Telford projected, for each end of the bridge, two battlemented towers including the cast-iron plates which clamped eight catenaries in a vice, each of them composed of five metallic cables. On the catenaries a metal structural work, on which a double wood layer created a passage, was placed. The better conditions of the wind and of the streams made possible a simpler structure of the Conway Bridge which, compared with the one on the Straits of Menai, presents a superior span.

The Bridge of Hammersmith was built between 1825 and 1827 by the engineer Clark and by Brown, an army captain. The two masonry piers, erected on the two shores and based on wood piles, contain the anchorages for the eight catenaries which represent the core of the structure. The sliding of the chains on the cylindrical rollers was used also for this bridge, as it is possible to understand from the plan reported in the figure. A wood-floor, proofed by using tar and pitch, ensured the practicability of the carriageways (fig. 2,3).

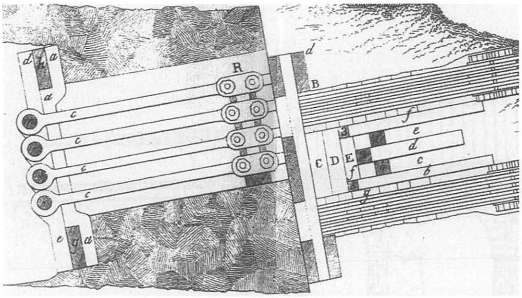

Figure 2
The Bridge on the Straits of Menai. A particular

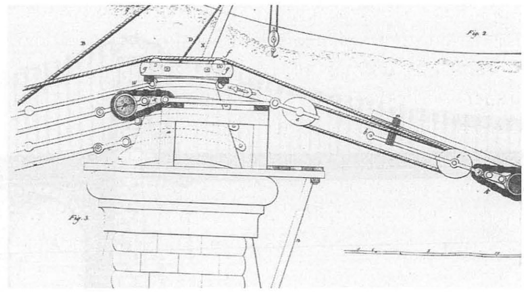

Figure 3
The Bridge on the Straits of Menai. Sliding of the catenaries on the piers

The construction of the Egyptian bridge in Saint Petersburg started in 1825 and ended one year later. It is characterized by a structure made up of catenaries, clasped to six Egyptian cast-iron columns and connected by a cast-iron beam containing the sliding rollers of the catenaries. All the columns and the cast-iron elements were decorated according to the style of the ancient Egypt, famous in that time both for the archaeological discoveries and for the Napoleonic military campaign. In order to antiquate the bridge, all the metallic parts were tinged with «bronze antique» colour. All the catenaries, housed in two cast-iron cables, extended to two granitic masonry blocks; the gap between the chain and the covering cable was filled with a mixture of wax and tar as a protection against humidity and corrosion. The carriageways of the bridge were made up of a wood double layer deck; the first layer was proofed by using pitch and tar. The bridge *of the Four Lions* and *of the Gryphons* are similar to the Egyptian one (fig.4).

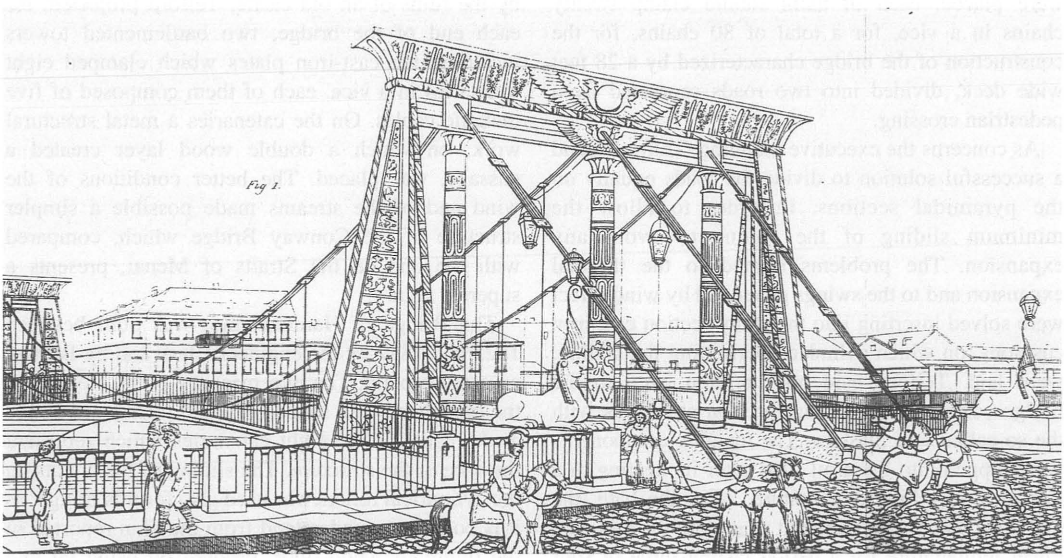

Figure 4
The Egyptian Bridge

All the above mentioned bridges, together with other examples of suspension structures, were built in Europe at the moment in which the construction of the suspension bridge on the Garigliano, near Neaples, was conceived. Since the old age, there was the necessity of a link between the two banks of this river, to make the connection between Naples and Rome easier. The first Bourbon sovereign who took into the right account the idea of constructing a bridge on the Garigliano was Ferdinand IV who, in 1788, commissioned his experts to carry out a research. The construction of a single span stony bridge was proposed to the king, but this project was never put into practise because of its excessive cost. Furthermore, the bad river floor conditions precluded the stability of the span abutments, which should have been liable to differential failures; even for this reason, in 1823 the construction of a suspension bridge was proposed to the king. This choice perfectly took place in the industrial set-up of the Reign of the two Sicilies: as a matter of fact, the iron production was an incentive for these people, who saw in the construction of the suspension bridge an opportunity, for the Reign, to attain the primacy for this kind of construction in Italy. The opportunity became true thanks to a young engineer, particularly skilful in planning, who, in December 1825, presented the king a research on the bridge and got from him the authorization.

Luigi Giura was born at Maschito, in Basilicata, on October 1st, 1795. He graduated in 1815 at the «School of Bridges and Roads», founded during the Napoleonic decennium, from which the prestigious school of Neapolitan civil engineering arose. According to his interest in the new techniques connected to the use of iron rising all over Europe, in 1826 he set out on a study journey to England and Germany, which gave him the opportunity to achieve the technical knowledges to start the construction of the bridge *Real Ferdinando*. According to the planning laws of that period, the executive planning had to comply with the following building rules:

- One or many chains are placed in the vertical planes of the bridgeheads; they form, in the upper area of the riverbed, that curve named catenary; they lay on a point on each of the two pillars erected at a given height on the banks; then they are driven into big stony blocks deeply laid underground.
- From the upside down arches of the chains, in such a way displaced, fall some vertical bars named hangers, which support the floor.
- Each chain is composed of three branches. The floor is supported by the branch placed between the two pillars and named of suspension. The other two branches, which start from the tops of the pillars and end in blocks, must hold the branch of suspension, and for this reason are called of restraint.
- The point in which the branch of suspension connects to the branch of restraint, that is to say the point where the chain touches the top of the pillars, is named point of suspension.
- The points of restraint are those where the restraint lines and their ends are driven into the walls.

Therefore the structure was organized so that the loads should rest on the suspension lines with tension, the maximum value of which, realised in the suspension points. The stresses were transferred from there to the branches of restraint so that they could be held by the masonry pillars in which the anchor plates were housed.

Luigi Giura paid particular attention to the suspension points, improving the techniques used by Brunel in London, to fit them on the bridge *Real Ferdinando*, marked out by two chains in a vertical plane. This particular point was realised through a pendulum made up of three big vertical links which allowed each branch of suspension and each branch of restraint to swing; this was made easier by a second pendulum, properly unrelated to the first, around the hinge of which the two above mentioned branches joined up.

Another important aspect studied by Luigi Giura is the hooking of the chains to the terminal plates of the catenary; besides the care for the constructive details, Luigi Giura worked so that the resultant of the tensions was always vertical. This was possible setting the pendulums in a perfectly vertical position and calculating the variations of load and temperature, which should be little determinative for the point of appliance of the resultant itself. The pendulums, placed inside the masonry piers, could easily swing because of their greater length compared with the hooking pins and this allowed to get a very low friction value. Therefore, the whole static system

gave the opportunity of getting a very low concentration of tensions at the piers level.

Two Egyptian style columns, surmounted by capitals ornamented with palmettes, placed on bases covered with freestone, were built on each of the banks. The columns bore four catenaries, each of them made up of two rectangular section bars, so that they formed the suspension line, in the shape of an upside down arch, and lines of restraint which, starting from the top of each column with an inclination of 28 grades, were clasped to stony blocks placed in underground passages, the entry of which was characterised by stony sculptures reproducing the sphinx. To the suspension catenary were connected 108 metallic bars, on which unloaded a slab made up of metallic rectangular section bars, used as a support for the wood-floor in oak crosspieces so as to build up three practicable ways, two of which were external walkways. The limestone used to shape the capitals of the piers had a 600kg/cmq pressure resistance, therefore it was quite resistant, like all the elements used to construct the bridge, to the effective acting loads.

The construction of the bridge ended in 1832 and was inaugurated by the King Ferdinando II himself; its cost was lower than the one estimated. The *Real Ferdinando* preserved a good state till the Second World War, when the Germans, retreating from Naples, set the central span off. By Giura's executive plans, kept at the Public Record Office, it was possible to estimate the total weight of the bridge at 260 kg/mq; the bridge structure was estimated to bear an 240kg/mq overloading with a chain stress of 500 tons (15 kg/mmq). It is necessary to add that, as concerns the static planning, the bridge had some defects, caused by the considerable deformation it was submitted to and for the lack of braces that compromised its resistance to wind and seism.

All the metallic structural work used for the bridge construction came from the industries of the Neapolitan Reign as token of an entrepreneurial and trade ability, never too much appreciated. The 70.000 kg of iron, used for the bridge construction, were made by the iron-foundries of the Prince of Satriano.

In the original Giura's drawings, the bridge was described with a wealth of detail. The voluminous fascicle, containing the plans signed by Giura and kept at the Public Record Office, is the token of technical choices, due to his own abilities and his being continuously abreast of the science; his technical choices were supported by laboratory tests which gave the opportunity to proportion both the chains and the hookings.

Nowadays the bridge has been restored with advanced techniques aiming at recovering the functional and performance levels of the end parts, still preserving a good state, and aiming at rebuilding the central part made up of aluminium. Therefore *the Real Ferdinando* keeps on holding the Italian technological primacy: it was the first suspension bridge in the past, it is the first aluminium bridge in the present.

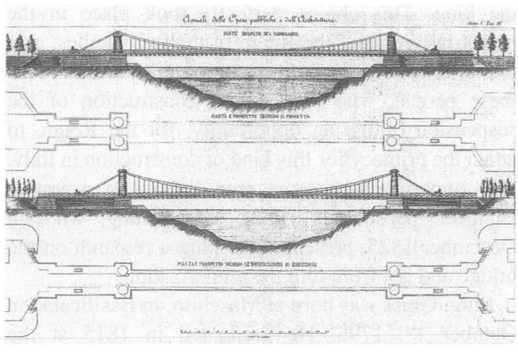

Figure 5
The bridge on the Garigliano, in Giura's drawings

Figure 6
The bridge on Garigliano. A view with the sphinx

Construction evolution of medieval tuscan monasteries: The tie beam system in the Sant'Agostino monastery at Nicosia (Pisa)

Federico Cecchetti
Mauro Sassu

Restoration of the static and functional configuration of the Nicosia religious complex, a monastery erected near Pisa beginning in 1258, has been studied with particular regard to the historical and structural problems associated to the masonry tie beams fitted to its walls. Entrance to the main building is through a courtyard containing the remains of earlier buildings, the church and old infirmary. A cloister forms the ground floor, while the first floor has been divided according to the layout typical of such buildings: many small cells opening off both sides of three wide corridors. A series of cellars also occupies the partly underground basement floor. The goals of the planned restoration operations are to consolidate the monastery, while seeking to preserve the structure's historical identity through accurate historical analysis and the use of traditional techniques (Del Bufalo, 1981–Dezzi Bardeschi, 1981). From this standpoint, the tie beam system appears particularly relevant and interesting, as it reveals the evolution (Manna, 1980–Nascè, 1982) of the various stages of maintenance performed on several parts of the monastery from the time of the Medici up until the addition of the latest reinforcements in 1985. The anchorage system is substantially the same for all the tie beams: one (or two superimposed) diagonal, square cross-sectional rods acting to retain a similarly square cross-sectional beam; the only tie beam with a circular cross section is the most recent one, inserted in 1985.

In order to guide the choices to be made in planning the operations and test their practicability, we have conducted accurate analyses of the beams' make-up via metallographic optical-microscope examination, from which it has been possible to determine the type of cast iron used (dating back to the 18^{th} century). Furthermore, analysis of the geometry of the various anchorage systems, supplemented by historical document research, has enabled determination of the successive stages of the monastery's construction and dating of the tie beams fashioned via traditional production techniques (Piccirilli et al 1994).

As it was impossible to execute tensile tests on iron specimens, a Vickers hardness test was used to determine the tensile strength values, while chemical and spectrographic analysis with the electron microscope confirmed that the beams could be joined via modern welding processes. Use of the minimally destructive technique of inserting a spinning cot in the middle of the beam has also been proposed in order to allow for regulation of the stress for safety reasons, i.e., to restrain the adjacent masonry walls during their disconnection and protect the original construction techniques used for the anchorages.

STATE OF THE ART-HISTORICAL REMARKS

Painstaking study of historical references, including bibliographical and cartographic sources, has allowed valuable materials to be uncovered for reconstructing the building's history, the alterations it has

undergone, the historical contexts in which it evolved, and the constructive methods and techniques in use during each period of its development (which, except for rare, fortunate cases, has turned out to be rather sketchy) (Giuffrè, 1990) .

Four basic stages in the monastery's lifetime have been identified (fig. 1):

— The 13th century (original complex);
— The 15th century;
— The 17th century;
— The additions made during the 18th to 19th centuries.

The original structure is visible for the most part on the monastery's eastern side, beyond the right arm of the church transept. A seemingly reasonable hypothesis holds that the original entrance to the monastery had to be further down on the southern side, in correspondence to the point of arrival on a roadway dating back to the late middle ages, still usable today.

Initially, the building's highest elevation was probably along a section of its western flank, as testified to by the mixed masonry bearing structure, which reaches this maximum height only along said segment. Instead, the height of the northern and eastern portions did not generally exceed the plane of the current mezzanine floor.

During the 15th century the access road to the monastery was restructured and widened, along with the original cloister, which was enlarged to its current layout.

The cloister was then made taller, through addition of a floor above the parvis; this probably called for the addition of structural reinforcements, as testified to by the large masonry columns built around the slender stone pillars, which were inadequate to sustain the added weight of the upper floor.

Written references of further additions to the

Figure 1a/b/c/d
Construction phases of the S. Agostino monastery in Nicosia (PI)

building are scarce: documentation exists (1876) only for construction of a new kitchen. However, considering the type of workmanship, materials and construction techniques used on the other additions, these were probably erected sometime during the 18^{th} and 19^{th} centuries.

Regarding the ties, the main object of our study, a first step was to record all those present in the monastery. From their state of maintenance and the presence of cracking on adjacent walls, a number of hypotheses were formulated regarding their static state (Scillone, Di Segni, 2000–Tassios, 1995). Dating them was performed via experimentation through application of non-destructive analyses performed on samples of the materials making up the ties. The results, supplemented by a brief bibliographical research, has made it possible to establish, though only as a rough approximation, that a good number of these reinforcements date back the period between the 18^{th} and the 19^{th} centuries.[1]

A survey of the tie heads, all of the «pole type», led to identification of three different types: the first characterized by very long, slender rods, with well-finished eyelets and very small tightening wedges (fig. 2).

Another type of tie is instead distinguished by much shorter, thicker rods, with a wedge-shaped extremity, probably for easing its insertion into the eyelet and whose tightening wedges are sometimes preceded by other metallic rods, with the function of tensing the tie and increasing the resistant section of the cross anchorage (fig. 3)

The ties on the northern façade belong to the third type encountered, whose geometry is intermediate between the two foregoing types, that is, they are quite long, and thick in cross section.

One interesting finding worth underscoring is the type of joint connecting the tie beams in the north-eastern area, at the height of the mezzanine level (fig. 4). This is a very long tie —made up of two square cross-sectional rods connected by means of a hook joint, which on one side is fitted to one of the pillars making up the system of arches along which it runs, while on the other, it is anchored to two different walls running parallel to each other. The area in which it is located was built up in the 17^{th} century, a building extension clearly testified to by the varying thickness and types of masonry used.

It can be hypothesized, therefore, that two different systems of tension beams have been applied: a first type dating back to the 18^{th} century, to add support to the walls of the old monastery complex, and a second one to sustain the building additions made in the 19^{th}

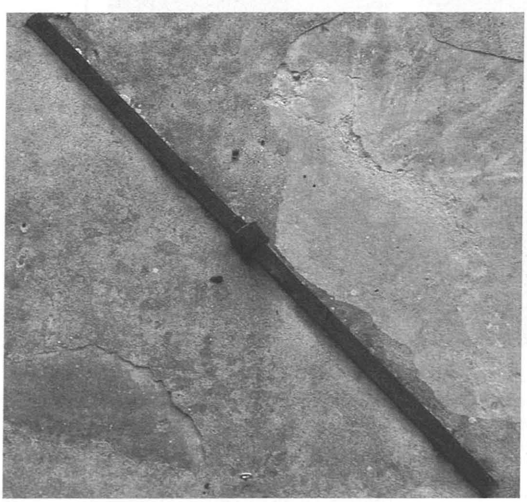

Figure 2
Tie beam anchorage (type n.1)

Figure 3
Tie beam anchorage (type n.2)

then cut and weld on a threaded sleeve, which will allow tensing and regulating of the tie. Thus, preliminary experimental metallographic and hardness testing was performed in order to determine the tie's historical value and check whether it could tolerate welding.

The unloading device. As designed, the mechanism absorbs (by the effects of friction) the stress resulting from disconnection of the rod, and transmits it —via the tension rods— from one extremity of the tie to the other. It should be emphasised that in order to render the device versatile, the tension rods adopted were threaded throughout their entire length, so that they could be adapted-via a system of nut and lock-nut to the various spans that may be encountered in various parts of the tie system of the monastery complex.

The tie-sleeve connection. In order to verify the feasibility of welding the new elements to the existing ones, a series of analyses has been conducted on the type of iron making up the existing ties. In order to avoid damaging the tie itself, this was performed by removing small portions of the metal and performing only those tests and analyses possible with such small samples. The samples have thus been subjected to the following:

— Brinell hardness tests;
— metallographic examination with the optical microscope;
— spectrographic analysis via the electron microscope;.
— chemical analysis.

The Brinell Hardness test is the only mechanical check possible on such small quantities of material. The tensile resistance of the constituent material of the ties, an indispensable parameter for planning restoration, can be deduced by virtue of a relation between the Brinell hardness and the limit tensile stress. The tests, whose results are reported in Table 1, were conducted in the laboratories of the departments of chemical (lab 1) and structural (lab 2) engineering.

Upon metallographic examination all the samples exhibited macro-inclusions at first visual inspection,; these are probably attributable to incomplete decarbonisation during the iron production process.

Table 1 Brinnell hardness test

Specimen	Lab. 1	Lab. 2	Brinnell hardness	Strength [Mpa]
A	149	143	146	438
B	123	133	128	384
C	85	89	87	261

The exams moreover revealed that the samples all have a rather large grain (see fig. 6), indicating a high concentration of impurities [2].

This exam enabled a first hypothesis regarding the metal's chemical composition to be formulated, in that observation by the optical microscope revealed that it is not composed of a two-element, iron-carbon alloy, such as steel, but soft iron with low carbon content.

Electron microscope examination confirmed the previously forecasted chemical composition of the metals examined: via spectrographic analysis of the digital images furnished by the electron microscope, we were able to determine the chemical composition of the analysed metal at a single point, as well as along a line and within a region.

The chemical analysis yielded the percentages of all elements present in the metal. The conclusions that can be drawn from the results of the analyses performed are that the tie beams are essentially made of a soft grade of iron, probably produced in an old

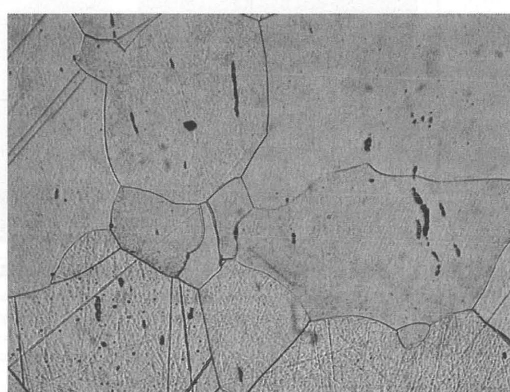

Figure 6
Metallographic view

iron mill by puddling[3] and can be welded perfectly only through forge welding.[4]

Iron with such a low concentration of carbon (Table 2, specimens A and B) is easily worked, and for this reason the rods in the monastery, if removed, could easily be threaded by lathing to allow direct insertion of the tensing sleeves, of course contingent upon verification of the beams bearing capacity.

Table 2 Spettroscopic test

Specimen	Carbonium	Silicium	Potassium	Sulphur	Tin
A	0,0100	0,1440	10,4157	0,0108	0,0094
B	0,0610	0,0970	10,3077	10,1212	0,0031
C	0,2160	0,2150	0,0046	0,0028	0,0086

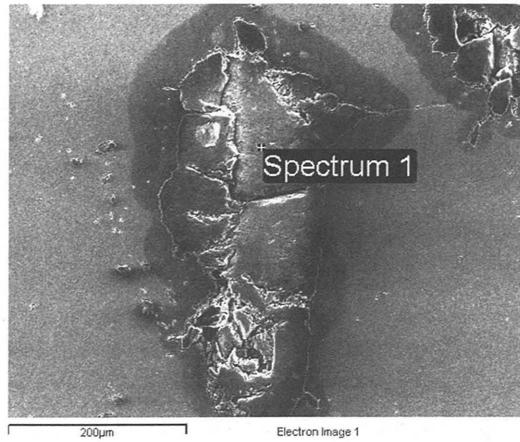

Figure 7
Iron impurity by electron microscope

The solution proposed instead calls for insertion of a threaded steel sleeve (Table 2, specimen C): the connection is to be made by angle bead welding. It should be noted that the temperatures attained in this welding process may provoke partial melting of the adjacent iron. In such situations, the considerable sulphur and potassium content of the impurities, could bond with the iron and give rise to composites that would weaken the connection. Therefore, the procedure must be planned with the widest possible margin of safety. However, it seems that such a drawback can be minimised if the welding is executed with particular care.

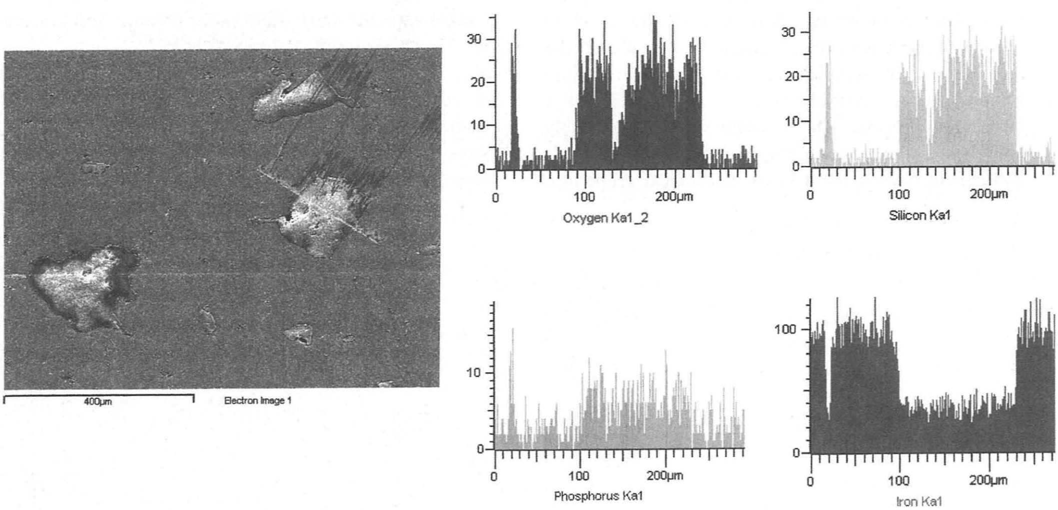

Figure 8
Example of spectrographic analysis on impurities

Conclusions

Many Tuscan buildings, in particular the numerous monasteries set on high ground or embankments, are often fitted with tension rods to reinforce their external walls. Researching their histories, including chemical-physical analysis, constitutes an important step in their safeguard and maintenance from the perspectives of both their static equilibrium and historical configuration. Moreover, any consolidation or reinforcement operations must be performed, as far as possible, through techniques that take into account the structures' geometric properties and consitituent materials.

Notes

1. The only indications come from the research conducted by Alessandro Del Buffalo for the History of Architecture course at the University of L'Aquila. An illustrated chart shows six different types of capichiave(?)head rod), each indicated with the period of its most wide-spread use. Cfr A. Del Bufalo in «Metodo storico di schedatura per interventi di restauro» drawn from «La Conservazione dei Monumenti-Metodologie di ricerca e tecniche di consolidamento contro il degrado», ed. Kappa, 1981, Rome, fig. 1 p. 172.
2. Actually, a certain variability was observed in the grain sizes. This is probably attributable to the different processes adopted in the ironworks that produced the rods. Some of the samples may in fact have a finer grain because during production they also underwent a lamination process, which would have considerably compacted the metallographic structure, with a consequent improvement in its mechanical properties, especially the tollerance for welding, which at the time was only done by forge welding.
3. With this production method the cast iron, set on a silica base, is heated at high temperatures and decarbonated by the air aspirated by the flame in particular types of forges. However, a fundamental aspect of the procedure is the puddling process, by which the product was stirred into a pasty iron mass, easily worked subsequently by drop-hammering/forging/pressing(?)maglio).
4. In this welding technique, used by blacksmiths to weld soft iron, the pieces are heated in an oven or forge until they take on a pasty consistency, and then hammered together.

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The marvellous timber trusses of XVIIIth century
The work of the German carpenter Schuhknecht

C. Ceraldi
E. Russo Ermolli

Nineteenth-century manuals and treatises when specially devoted to art of carpentry, like the work of Krafft (1805) or of Emy (1841), as well as when investigating the whole building theory and practice like the treatise of Rondelet (1810), paid great attention to timber trusses of large span. In fact, illustration of past and current technical capabilities of carpentry as guides to professional exercise was one of principal aims of those authors.

Between the most quoted examples of large span timber trusses there are the covering structures of the «exercise halls» which were built at the end of XVIII century to allow military exercise during winter. These structures, together with some examples of the beginning of XIX century, as that designed by M. Betancourt and built in Moscow in 1818, are the apex of carpentry art before new technologies and new materials provided a different way of covering large spaces.

In particular, two roof trusses designed by the German carpenter Schuhknecht show the high achievements of carpentry art, because of the large span, the structural conception and the technical arrangements:

- the covering structure of the exercise hall in Darmstad (Germany), built in 1771–1772 and demolished in 1892 to allow the construction of the Landesmuseum, with a span of about 44 m;
- the covering structure of an exercise hall to be built in Moscow, designed in 1781, but never constructed, with a span of about 84 m.

A geometrical scheme of those structures, based upon historical surveys, has been derived to study their structural behaviour; then the structural scheme has been deduced paying great attention to technological solutions designed for internal joints. The aim of this structural analysis has been not only the evaluation of the reliability of their design, but also the understanding of the carpenter structural intuitions on which this design is based. An enlightening factor is the critical review of the interpretation and judgement of these constructions couched by authors of famous and widespread treatises like Rondelet and Emy. Consequently this analysis gives also the opportunity to understand their structural ideas about timber trusses.

Darmstadt Exercise Hall

Historical news and sources

Manuals sources devoted to illustration of architectonical production and specifically of timber carpentry agree in indicating the covering of Darmstadt exercise hall, built in 1771, as the example of greater relief in the eighteenth century production. To its construction took part Johann Jacob Hill in quality of chief-engineer, and carpenter Johann Martin Schuhknecht as works-director. They produced numerous varying designs in relation to the

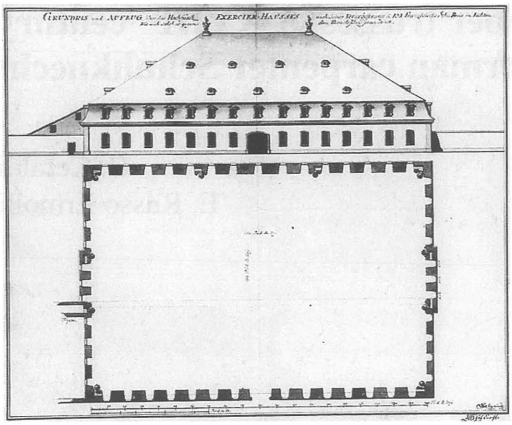

Figure 1
Early version of Darmsatdt exercise hall (Historical Archives of Darmstadt)

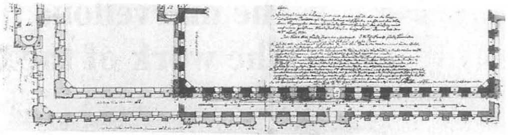

Figure 2
Partial plan with enlarging proposals (Historical Archives of Darmstadt)

changing demands of German regional state Assia-Darmstadt Landgrave, Luigi IX.

One of those design, drawn up by Hill and Schuhknecht, and dated November 1770, is illustrated in Figure 1 by the Paraderplatz prospect and the hall plan, which is quoted with inner dimensions of 130 × 200 feet (42,25 × 64,50 m).

Works began in coincidence of the carnival of year 1771 and during construction an ulterior widening was decided, carrying the inner dimensions to 130 × 272 French feet (42,25 × 88,40 m). The design of those modifications, by Hill and Schuhknecht, is shown in a coloured ink drawing, where the original profile, some temporary hypothesis, and the final choice, which is the outer one, can be seen, Figure 2.

The confirmation that this was the finally realized plan can be obtained by various recordings and drawings stored in the historical archives of the city of Darmstadt, as the nineteenth-century survey, containing the Paraderplatz and the east prospects, the transversal and longitudinal cross-sections, and plans on several levels, reported in Figure 3.

The importance of this work and its oneness in the panorama of contemporary constructions is testified by comments in writings of that age, where great admiration for its realization is manifested. H. W. von Gunderode in 1782, in *Neuen Fragmenten zur Kenntnis des Menschen*, expresses great wonder for

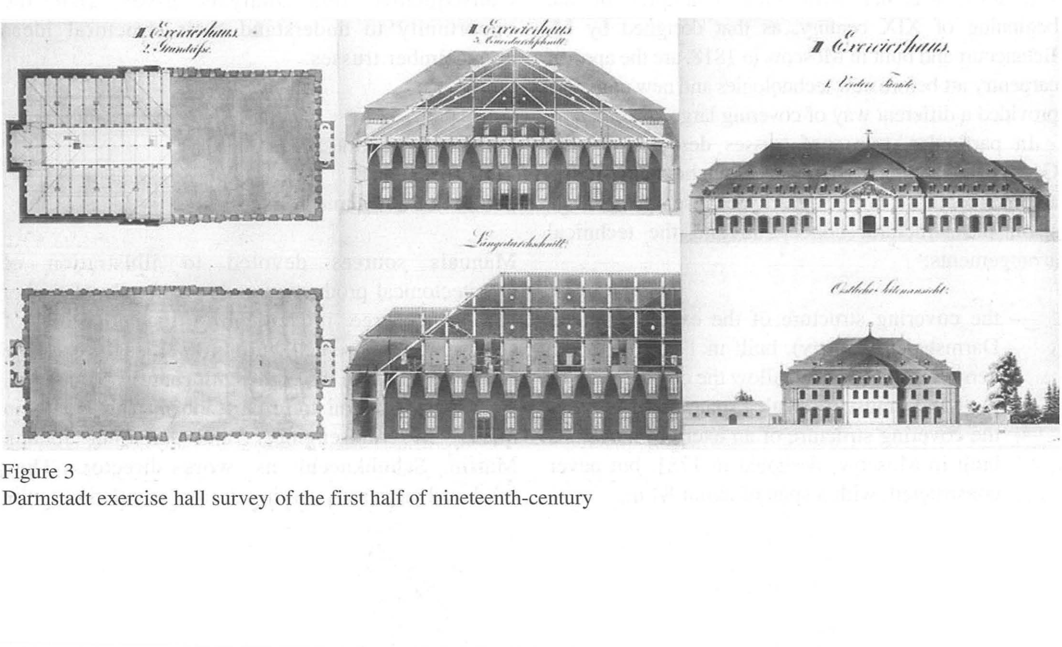

Figure 3
Darmstadt exercise hall survey of the first half of nineteenth-century

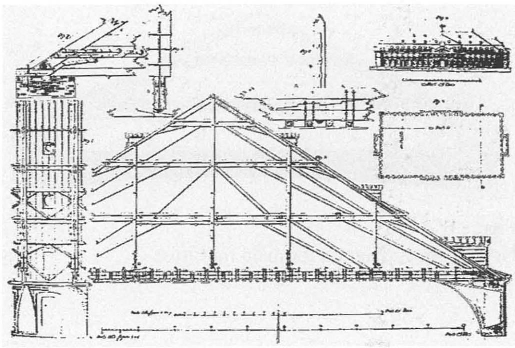

Figure 4
Table 112 from the Treatise of Rondelet ([1810] 1833)

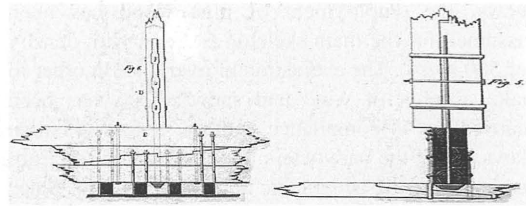

Figure 5
Detail from table 112 of Rondelet

the enormous dimensions of the hall, completely free from intermediate supporting pillars (Worner 1882).

Also Rondelet ([1810], 1833) testifies the importance of this work bringing it back in Table CXII of his treatise, Figure 4. The comparison with the nineteenth-century survey, Figure 3, shows the total coincidence of the structural system.

A third representation of this structure exists in literature, supplied by Emy in Table 92 of his handbook (Emy 1841). This valuable timber covering unfortunately can be no longer admired as it was demolished in 1892 to allow the construction of Landesmuseum.

Description of the structure

Through the cited documentation, defining in details the dimensions of the construction and its structural outline has been possible. It is a roof truss conceived on the principle of triangulation, formed by the tie-beam and the rafters, and also three collar-beams and five posts. Moreover the scheme is enriched by diagonal struts which, in the designer's conception, had to collaborate with the principal rafters. Tie-beam, whose total length is of about 46 m, is formed by three orders of overlapping beams, of section 0,26 × 0,28 m each, jointed in longitudinal direction with rack-like superimpositions. The cross-sectional connection would have to be guaranteed by the metallic strips disposed alternatively on two sides of the tie-beam, also supporting the ceiling, Figure 5.

The posts behave like true double hanging posts, formed by two elements of cross-sectional dimensions 0,32 × 0,125 m each, which embrace the collar-beams, mutually connected by metal pulling. These collar-beams generally are constituted by braces of beams separated by spacers in wood. The principal rafters, of cross-sectional dimensions 0,20 × 0,26 m, are coupled to roof struts of transversal dimensions 0,10 × 0,15 m, until the quota of the upper collar-beam. In the upper part only the roof strut continues. The roof trusses, placed at a mutual distance of 3,90 m, are spaced out by seven secondary trusses, equidistant, composed only by roof struts, of section 0,10 × 0,15 m. Principal roof trusses and secondary ones are connected by transversal elements, disposed in the roof slope and in correspondence of collar-beams, guaranteeing an overall spatial behaviour. The top of the roof trusses is at a level of 14,21 m.

Structural verification

The structural behaviour has been studied constructing a spatial scheme, formed by one-dimensional elements connected so as to reproduce the joints typologies deducible from the consulted documentation. The analysis has been led in linear elastic range.[1]

To define loads acting on the structure, a covering system widely used at the end of the eighteenth century in central Europe has been assumed, as the real one is not described in the consulted sources. It is constituted by a mantle of zinc slabs, 3 mm thick, fixed on a fir table layer, 40 mm thick. Lacking sure

news, the employment of pine wood has been assumed for the main skeleton essence, with density of 500 kg/m^3. The evaluation of overloads in order to take account of wind and snow effects has been carried out in compliance with the enforced Italian laws, using the parameters brought back in the Euro code 1,[2] for the geographic area of Darmstadt. Since the more onerous load condition turns out that one obtained in presence of snow overload alone, wind has not been taken in consideration. For snow, reference has been made to German Zone II, for an altitude smaller than 200 m (Darmstadt is 146 m). In dead loads evaluation, the presence of the ceiling, constituted of fir joists of cross-dimensions 0,18 – 0,18 m, 0,53 m spaced, and of a timber layer 40 mm thick, is relevant.

Numerical simulation has revealed that critics moved by Emy (1841) to the design of this grand structure turn out partially founded: in fact he observes that the employed amount of timber is more then the strictly necessary one and some structural elements aren't well disposed. In particular he points out that diagonal struts contribute to load the joggle more than to support it. Such diagonal elements, in the structural conception of the age, had to supply intermediate supports to collar-beams. Numerical analysis shows that, nearly in totalities, the diagonal struts are in tension, when using a structural scheme in which bilateral mutual end-bounds are allowed. But the designed mutual bounds, simple timber on timber notches, do not allow traction transmissions, so it is truth that diagonal struts constitute only an overload for the structure.

Normal stress and bending moment diagrams of the final structural scheme, in which all previous observations have been taken into account, are then obtained for the main roof trusses, Figure 6 and Figure 7.

These stresses induce in the principal structural elements the following maximum local stresses:

— rafter	11,33 MPa
— tie-beam	7,95 MPa
— diagonal strut	7,53 MPa
— collar-beam	12,04 MPa

These values could be considered allowable for a first class pine wood essence ($\sigma_{all/bending}$ = 12 MPa), using the allowable maximum strength criterion.

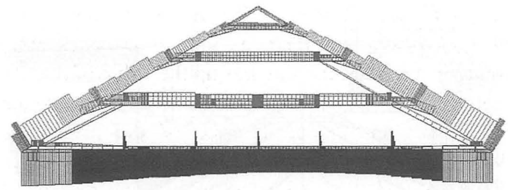

Figure 6
Normal stress diagram for main roof truss

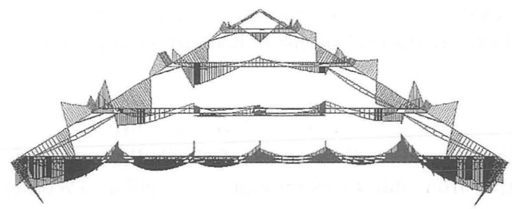

Figure 7
Bending moment diagram for main roof truss

With reference to secondary trusses, formed only with roof struts, the chosen schematization, which implies load distribution between principal and secondary trusses, allowed by purlins action, and shows the presence of simple supporting conditions at the inferior ends of roof struts, leads to an high value of stress (13,39 MPa) near the ends. This value however is not likely to induce structure collapse.

Comments to results

The timber structure of Darmstadt exercise hall constitutes the first example of covering with a span larger than 40 m, whose existence is widely documented and which would have resisted until nowadays if urban planning transformations had not demanded its demolition.

In spite of critical observations expressed by Emy in his treatise, it's impossible to agree with Rondelet assertion (Rondelet [1810] 1833), which defines such wonderful work of carpentry one confused combination of secondary elements. In fact stresses found in single structural elements demonstrate a correct distribution of wood material, without

particular super-dimensioning if not in the double hanging posts for which the maximum stress, only found in few sections, catches up the value of 5 MPa.

Moreover the employed amount of wood is not particularly excessive, in spite of the ineffective diagonal elements presence. It is observed in fact that the cubature of wood used for square meters of covered surface turns out equal to 217 dm^3/m^2, and is inferior to the values found for other large timber covering structures of the past, as for example the roof trusses of the church of S. Paul outside the walls in Rome, IX century, with about 24 m of span, and a cubature value of 314 dm^3/m^2 (Ceraldi, Mormone and Russo Ermolli 2000).

Moscow Exercise Hall Described by Krafft

Historical news and sources

Manuals sources agree in indicating the covering of Darmstadt exercise hall as the example of greater importance between eighteenth-century carpentry productions.

The great admiration which this work induced near the contemporaries gave place, according to Krafft (1805), to the design of an exercise hall to be built in Moscow, with the considerable span of about 84 m. In fact, commenting Table 39 of his collection, in which drawings of that structure are reproduced, Figure 8, Krafft writes:

> Paul I, empereur de toutes les Russies, faisant son voyage d'Europe en 1781, visita divers édifices, comme chateaux et habitations particulières, et s'attacha principalement à l'examen des constructions militaires; il fut extremement surpris en voyant la grande salle d'exercice à Darmstadt, dans la quelle on fait manoeuvrer les troupes pendant l'hiver pour ne pas interrompre leur instruction. Il dèmanda aussitot que les artistes s'occupassent du project d'un manége pour etre construit à Moscow, le quel serait exécuté sur un terrain de 1800 pieds de longueur et 290 pieds de largheur hors des murs, et aurait 220 pieds de large dans centre. Il désirait aussi que l'on y menageat une galerie pour les spectateurs et pour y loger le conduites des feux pour le chauffage de la salle. Un project qui réunissaid ces conditions lui fut présenté par un maitre charpentier d'Allemagne, et l'empereur le fit exécuter en 1790; il sert de salle d'exercice pour la cavallerie et l'ifanterie des cosaques (Krafft 1805).

It could therefore be deduced that the German carpenter to which Krafft alludes is just Schuhknecht, the skilful designer of the structures of Darmstadt exercise hall.

Also Rondelet, in the first edition of his treatise (Rondelet 1810), in Table 116, Figure 9, reproduces the drawings of this carpentry, introducing on the right side, an improving proposal modifying the structural scheme.

It turns out that such carpentry was never constructed. In fact Rondelet, in the last edition of his treatise (Rondelet [1810] 1833), in the paragraph devoted to the description of the Moscow exercise hall really built in 1818, by M. Betancourt, reports his

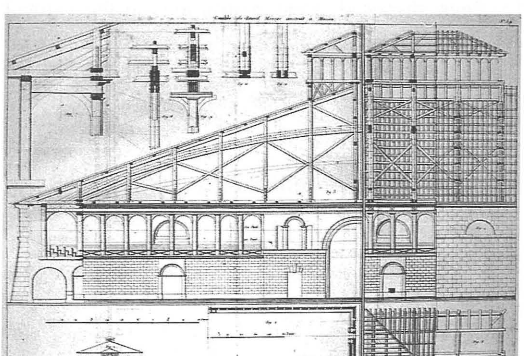

Figure 8
Roof truss of a great covered riding-ground in Moscow (Krafft 1805)

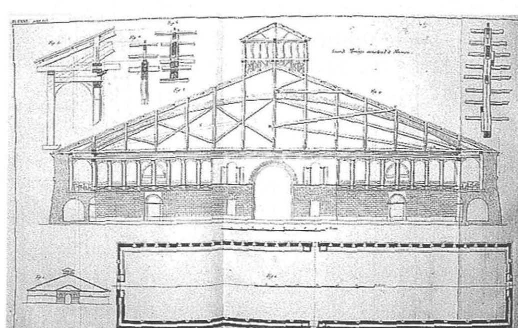

Figure 9
Table 116 from the first edition of Rondelet's treatise (Rondelet 1810)

words to testify that Emperor Alexander I asked for various designs in order to build an exercise hall in Moscow, as in this city none has been built yet. To such purpose Rondelet annotates that the building represented in Krafft collection never was constructed, correcting what he has asserted in his treatise first edition. Moreover in the last edition, Rondelet returns on the structural inadequacy of this design, proposing, on right side of Table 115, Figure 10, a new improving hypothesis.

Another description of the design for Moscow exercise hall, can be found in Emy's treatise, where in Table 93, are reported the original design of the German carpenter as well as the modifications proposed by Rondelet.

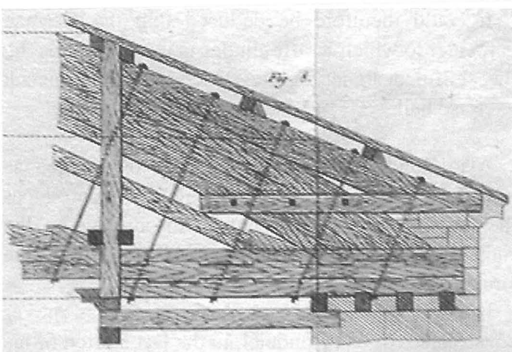

Figure 10
Detail of Table 115 from Rondelet's treatise 6th edition (Rondelet [1810] 1833)

Description of the structure

The drawings supplied by Krafft (1805) and illustrated here in figure 8, are believed more likely the originals ones as they represent the original source for the other writers. From these drawings it can be drawn that the hall has external total dimensions of 1.265 × 285 French feet (411 × 92,50 m) and inner clear dimensions of 1.238 × 260 F. f. (402,50 × 84 m). Along perimeter walls on the inner side a double order of galleries is placed: the lower one, of masonry, is devoted to the lodging of boilers and of all the apparatuses for hall heating systerm; the upper one is constituted of a timber frame, in adhesion to the perimeter masonry, and receives the tiers of seats for the audience. The inner dimensions, clear of the gallery, turn out equal to 1.200 × 220 F. f. (390 × 71,50 m).

The hall is lighted by the semicircular windows placed at the level of the upper gallery, in number of 32 for each longitudinal prospect, and also by rectangular shaped skylights, placed on the covering along the central axis. The carpentry of these skylights is also illustrated by Krafft in Table 39, Figure 8, (Krafft 1805).

The timber structure of the covering has triangular shape of about 22° slope, and is constituted by 68 trusses placed at a mutual distance of 5,75 m. The main support of each one is a timber arch formed of three ranks of overlapping and teeth-shape connected pieces, withhold with iron bolts and strips. The principal rafters carrying the roof, as well as the big tie-beams, are supported by sturdy double hanging posts, withhold by large struts with a Saint Andrew cross disposition. The double hanging posts embrace the principal rafters, the arch and the bracings, and sustain the tie-beam for means of metallic strips. The tie-beam is constituted of one single beam with longitudinal Jupiter dart joints. Tie-beam divides in two elements where it meets arch extremities, allowing direct connection of the arch with the frame of the gallery.

With the exclusion of the area under the skylights, the whole hall is covered by a ceiling, built with a timber layer brought up by transversal lumbers.

Upon principal rafters, square purlins are fixed, which, together with Saint Andrew cross-shaped bracings, disposed along central axis in the vertical plane, guarantee the transversal connection between the roof trusses. The structure is completed by roof struts, in number of nine for each inter-space.

Critical appraisals in the nineteenth-century literature

Rondelet's judgment on German carpenter's design is strongly critical. In fact he observes that although its fascination, the truss would turn out too weak for being able to support the enormous total load to which it would have been subjected. In the first edition of his treatise, he proposes the modifications shown on the right side of his Table 116, Figure 9

(Rondelet 1810). In these proposals of structural changes, the concepts on which Rondelet based his ideas of structural optimization in carpentry designing can be seen, that is the creation, inside the main scheme, of substructures hardening the main structure and supplying a better distribution of loads. In later editions and in particular in the sixth one, Rondelet brings more modifications to the structural scheme, previewing the strengthening of the arch, which passes from three to five orders of elements, and of the tie-beam, which is tripled in thickness. In this way he concentrates the absorption of the arch trust all in the arch —tie-beam joint, eliminating the supporting timber frame and any interference with the perimeter masonry (Fig. 10).

Structural verification

The structural scheme verified is that corresponding to the original drawings quoted by Krafft (1805). This scheme reproduces one of the roof trusses characterized by the presence of the skylight, as load condition is heavier, and has been deduced and calculated with the same procedure followed for Darmstrad scheme. Wind and snow overloads evaluation has been done in compliance with the enforced Italian laws, using the parameters brought back in the Euro code 1,[2] for the geographic area of Helsinki.

To define external bounds and specifically the interaction between roof truss and supporting walls, the presence of the upper gallery timber frame has been taken into account. This frame is constituted of a couple of sturdy pilasters of cross sectional dimensions 0,70 × 0,80 m, and of couples of transversal struts solidly jointed with the arch. The whole forms the main supporting structure of the roof truss and transfers its trusts to perimeter walls. In the calculation scheme, this situation has been modelled assuming for the pilaster adherent to masonry the structural hypothesis of elastic leaning continuous beam behaviour. The only structural elements of the roof truss which directly lean on the walls are the roof struts.

Calculation of this scheme has shown that the covering structure pulling effect gives rise to perimeter walls overturning. So a different scheme has been drawn, without masonry walls contrasting effect on timber frame pilasters, as this contrast can't be present any more when masonry, from a theoretical point of view, looses its bearing capacity. Comparison of results coming out of the two structural hypotheses is shown in Figures 11, 12.

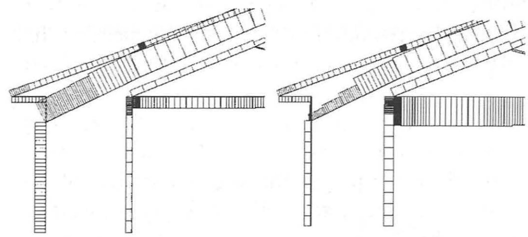

Figure 11
Comparison between normal stresses diagrams with and without elastic support

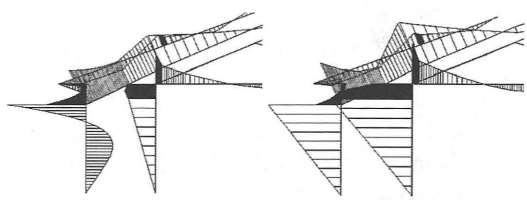

Figure 12
Comparison between Bending moment diagrams with and without elastic support

When masonry counter pull is absent, arch normal stress is obviously lower, while bending moment in the internal pilaster of the timber frame increases considerably. Nonetheless pilasters cross sectional dimensions allow supporting those trusts, with a maximum stress of 7,62 MPa.

Complete normal stress and bending moment diagrams corresponding to the second calculation scheme are reproduced in Figures 13 and 14.

With reference to these trusts, it can be verified that the strength of supporting timber frames, never took into account by Rondelet, can bear the high roof truss pull, even in the limit case, corresponding to masonry walls overturning.

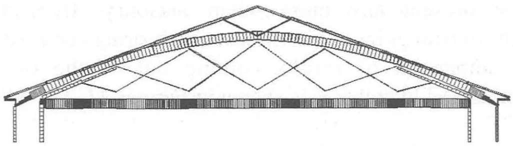

Figure 13
Normal stresses diagram of the scheme without elastic support

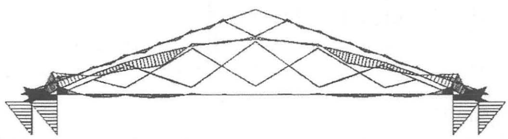

Figure 14
Bending moment diagram of the scheme without elastic support

Maximum stresses, induced in the main structural elements, are:

— arch 8,55 MPa
— tie-beam 12,96 MPa
— pilaster 7,62 MPa

These values could be considered allowable for a first class pine wood essence ($\sigma_{all/bending}$ = 12 MPa), using the allowable maximum strength criterion.

Comments to results

This extraordinary timber structure, with its clear span of about 84 m, shows the designing ability of its creator, who reveals himself as a skilful carpenter. To avoid the employment of a tie-beam of excessive cross sectional dimensions, he distributes the enormous arch pull between the tie beam and the supporting frame, which is an integrative part of the whole structure.

Moreover the amount of wood used for square metres of covered surface is not excessive, if compared to that of Darmstadt covering structure. In fact the latter is only 90 unity less than Moscow covering structure, which, with a span quite double of Darmstadt, has a cubature value of 307 dm^3/m^2. None of the main structural elements is really over dimensioned, and also the double hanging posts, with their total cross sectional dimensions of 0,69 × 0,48 m, support a maximum stress value of 8,2 MPa. Some reservations can be expressed only about diagonal struts, as their maximum stress values are very low, and only those disposed near the supporting frames reach an acceptable stress value of 5,3 MPa.

Concluding Remarks

The study of this marvellous timber structures evidences the high levels eighteenth-century carpentry art had reached, in designing as well as in building. Great attention in technological detail designing can be appreciated, with large employment of metallic elements, necessary to guarantee bonding strength of joints between timber elements making one structural component, as well as strength of structural internal joints. For example, the tie-beam of Darmstradt roof truss is built with three orders of lumbers, whose transversal cooperation is guaranteed by metal plate inserted between the two elements of the double hanging post, fixed at the lower end by an iron dowel.

Another cause of great wonder comes from recognising in these structures a conspicuous sensibility in grasping global structural behaviour, in an era in which theoretical study, even if it had reached important targets, as in works of Jacob e Daniel Bernoulli, and also of Euler, was not the common cultural background of architects, and of carpenters. At the beginning of XIX century, in spite of Navier's synthesis between theory and structural designing, structural concepts for scantling of such complex structures weren't available. So Rondelet and Emy, when judging structural behaviour of these timber masterpieces, concentrate on technological details, limiting the theoretical aspect to pure intuitive understanding. The practical aim of their works drives them to dwell upon the description of analyzed constructive details and of changing for the better proposed by themselves.

In conclusion, nowadays studying of this wonderful timber structures evidences the role of

skilful carpenters as Johann Martin Schuhknecht in developing timber construction technique.

Notes

1. The software used is Nolian program by Softing.
2. *Basi di calcolo ed azioni sulle strutture*, 2–3 Azioni sulle strutture, carichi da neve. UNI ENV 1991. 2–3 October 1996.

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The Swiss covered bridges of eighteenth century A special case: The bridge of Schaffhausen

C. Ceraldi
E. Russo Ermolli

In history of timber carpentries, great interest comes out of Swiss covered bridges from a technological as well as a structural point of view. Beyond the numerous testimonies of such still existing works, acquaintance of the most important realizations is offered by the consultation of handbooks on carpentry, like those of Krafft (1805) and Emy (1841), and by widespread works as Rondelet's treatise (Rondelet 1810).

The bridge object of the present study constitutes a singular case as it is reproduced by some treatise writers as a bridge realized in the city of Wettingen, on Limmat River, but in truth it was never constructed. It is instead only the first design, set aside later on, proposed by Hans Ulrich Grubenmann for Rein river crossing at Schaffhausen. The interest for such design is motivated by the extremely dared structural conception: the potentialities of the structural scheme composite with struts arranged in the vertical surfaces aside of the track, already adopted in many illustrious examples of the past, come exalted through the connection of such structures with those of a covering central skeleton, realizing an effective spatial scheme.

A calculation scheme, based on the reproductions of the original design, and on a precise reconstruction of technological solutions adopted at the end of eighteenth-century in Switzerland, has been conceived to verify the reliability of this design which, with a free span of about 120 m, represents the attainment of a limit never equaled.

Structural evolution of covered bridges until the eighteenth century

Covered bridges can surely be included between the most fascinating timber structures of the past, and in particular those constructed in Switzerland in the XVIII century. The desire of protecting the main timber structures from the atmospheric agents, especially in the alpine regions, pushed the constructors of timber bridges to adopt covering systems of wood tables or tiles, and often wood tables were placed side by side giving rise to partially or totally blind vertical walls, at the aim of protecting against wind action too.

Between the known ancient Swiss covered bridges, there is the Kapellbrücke in Lucerne, built in the beginning of thirteenth century, which has a total

Figure 1
Kapellbrücke in Lucerne

length of 285 m, composed with spans of 7,65 m, and is supported by a simple beam structure, Figure 1.

This bridge, destroyed by a fire in 1993, has been then reconstructed in recent times.

A beautiful example of Italian covered bridges is the famous bridge of Bassano del Grappa (VI) of 1561, on Brenta River, built by Palladio (1508–1580), whose track is supported by an underlying structure. The spans, about 12,00 m long, show a trestle structure with inclined struts, Figure 2.

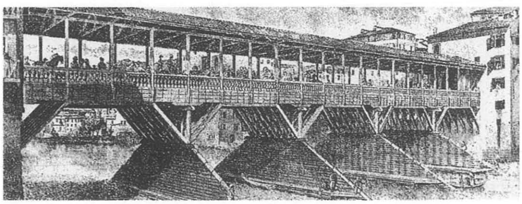

Figure 2
Bassano Bridge in a nineteenth century press

The bridge, destroyed in 1945, has been reconstructed identical to the original one in 1948.

However these support solutions with spans of modest length, don't represent, in the structural evolution of covered bridges, the most interesting examples, which can instead be ascribed to a different structural typology characterized by the presence of bearing elements disposed mainly above the track, besides those standing below.

Such typology can be interpreted like a different version of bridges «with lower deck», with bearing structures all disposed above the track, in contrast to bridges «with upper deck» in which the bearing structure is developed completely under the floor. The lower deck typology is that which Palladio develops and reproduces in book III of his treatise *The four books of Architecture*, published for the first time in 1570. Here various designs of bridges, based on the structural scheme of roof trusses, are proposed, all with an upper truss, Figure 3.

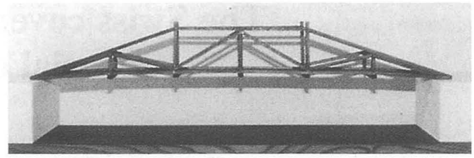

Figure 3
Timber model of the bridge on Cismone River

An interesting anticipation was that one of Leonardo from Vinci (1452–1519). In «folio 23», Code B he represents two bridges which can be classified as lower deck ones. In the first one, moreover, one observes how the principal structure, contained in the vertical surfaces aside the track, develops upon as well as below the floor, with inclined struts pushing on the banks, Figure 4.

Leonardo develops and synthesizes structural ideas already partially expressed in medieval age, as it is testified in «folio 20» of the Note-book of Villard de Honnecourt (XIII century) in which the author illustrates the modalities of building a bridge, based on the disposition of underlying inclined struts (Russo Ermolli 1995), Figure 5.

The employment in lower deck bridges of pushing structural elements, disposed also under the track, realizes a composite structural scheme which is

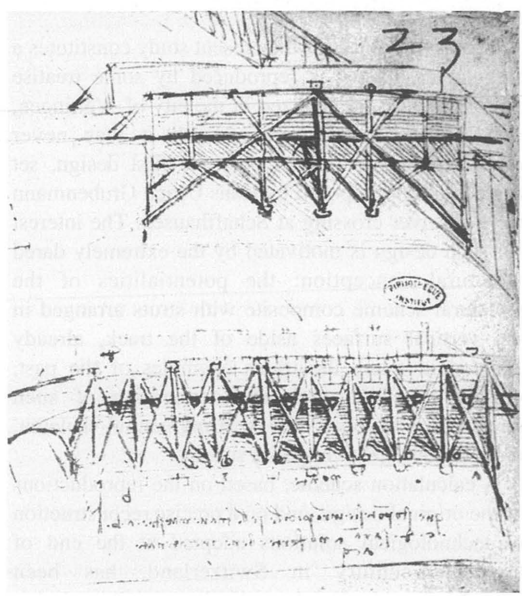

Figure 4
Leonardo's bridges in «folio 23, Code B»

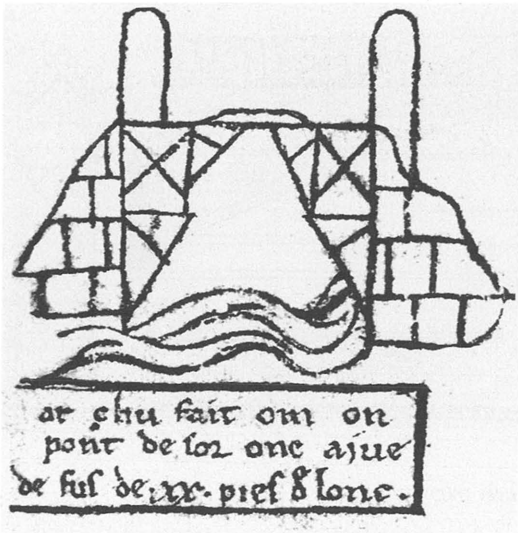

Figure 5
Bridge from the Note-book of Villard de Honnecourt

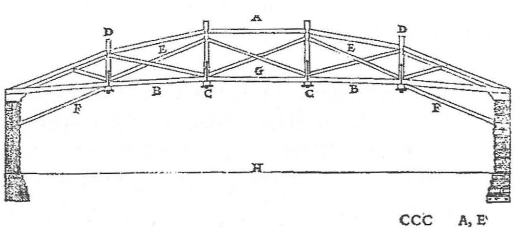

Figure 6
Palladio's «Second invention»

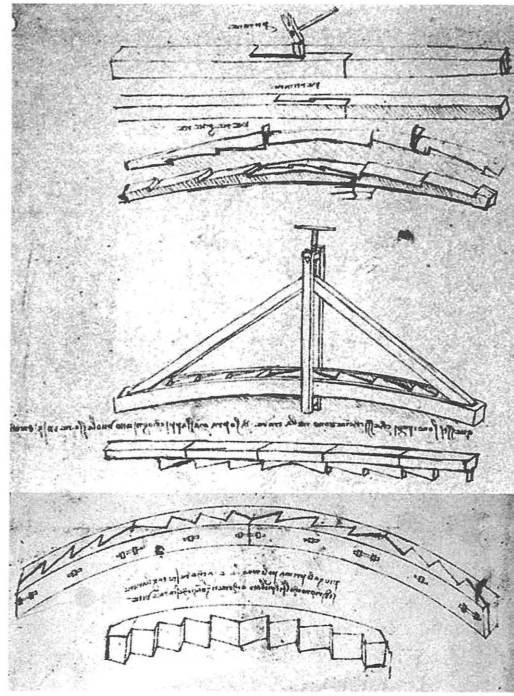

Figure 7
Atlantic Code. Folio 33 v. b and Folio 344 v. a

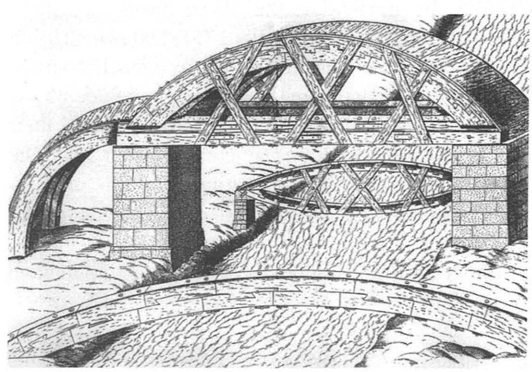

Figure 8
Drawing of a bridge from Veranzio's «Machinae novae»

obscurely present also in the «second invention» of Palladio, Figure 6.

This solution aims at transforming one simply supported structure in a complex of pushing type. Such structural behavior is a clear anticipation of the timber arch, whose structural effectiveness already had been understood, but whose practical realization was delayed by technological difficulties. Attempts to connect timber elements, making them behave like a single structural system, resembling an arch, had been proposed by Leonardo in the Atlantic Code, Figure 7, and by Veranzio (1551–1617) in his work «Machinae novae» of 1595, Figure 8.

The constructive problem of making timber arches of great span was resolved by Swiss carpenters in the

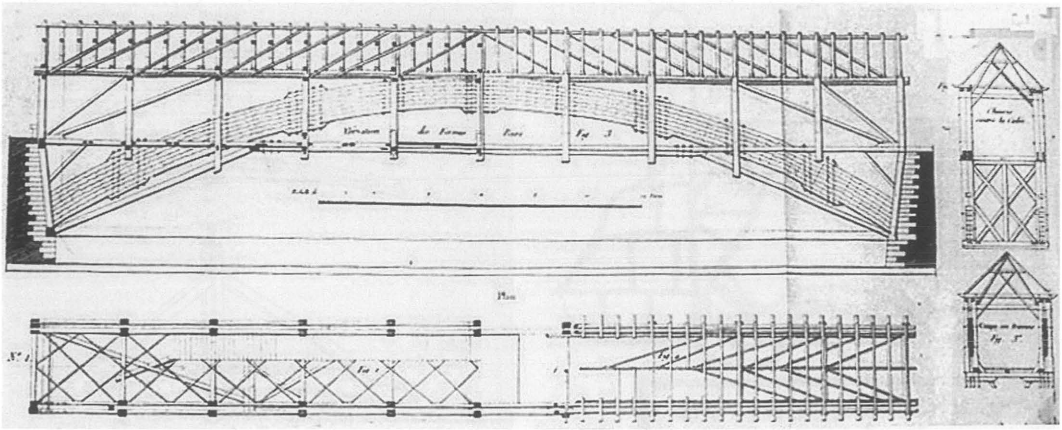

Figure 9
Wettingen Bridge (1765) from Table 28 of Krafft's Treatise (Krafft 1805)

eighteenth century, employing timber table notched and bent, held with iron bolts placed in correspondence of hanging double posts, Figure 9.

Such building system, which allowed exceeding spans of 30 m easily, first developed side by side and therefore substituted the composite structural scheme with inclined struts. Both constructive typologies were skillfully used by carpenters of Grubenmann family. In particular the three brothers Jacob (1694–1758), Johannes (1707–1771) and Hans Ulrich (1709–1783), born in Teufen in Appenzell Canton, built the most pregnant examples of eighteenth — century timber architecture in Switzerland and their work was held in so great consideration near the contemporaries and the nineteenth— century researchers to give place also to legends (Blaser 1982).

Between the existing bridges built by Grubenmann brothers, the following ones can be remembered:

- the most ancient bridge built by Johannes, the Rümlangbrücke near Oberglatt, dated 1766–1767,with arch typology and a span of about 28 m;
- the Kubelbrücke, near Herisau and Stein, built by Hans Ulrich in 1778, with multiple hanging trusses without nails or iron dogs and a span of about 30 m.

However the bridges which gave greater notoriety to Grubenmann family unfortunately had been destroyed during the Napoleonic wars, by French troops, in 1799. They are:

- the Schaffhausen bridge on Reno river, built in 1755–1758 by Hans Ulrich, with two spans of 52,00 and 58,80 m, constructed in fir with composite truss frames and inclined struts, Figure 10.
- the Wettingen bridge on Limmat river, built by Hans Ulrich in 1765, with a span of about 61 m, constructed with two sturdy arches constituted by notched and bolted overlapping beams, Figure 9.

A proof of the importance attached by contemporaries to these Grubenmann's bridges is the presentation of their plan, sections and prospects, made at the «Academie Royale d'Architecture», in 1771 by J. P. Blondel, who, it must be remembered, has been an active collaborator of the Enlightenment «encyclopedia» writers (Navone 2002).

But the work which for its boldness by far exceeds the quoted examples, constituting the apex in timber carpentry art, is represented by the first design Hans Ulrich Grubenmann made for Reno river crossing at Schaffhausen with only one span, a design never

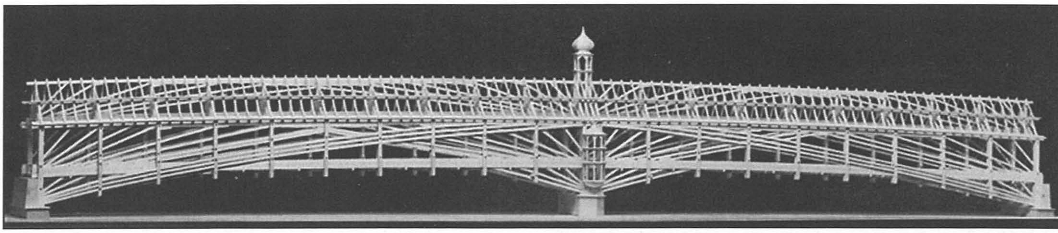

Figure 10
Model of Schaffhausen bridge with two spans (Steinmann, 1984)

realized. To its place it was constructed instead a bridge with two spans, Figure 10.

Historical news and sources

Steinmann (1984) reports that in 1775 Schaffhausen Town Council commissioned to Grubenmann master-carpenter the design of a timber bridge on Reno river, substituting the masonry one collapsed. The first design contemplated only one span of 120 m, which compelled the Communal Administrators to distrust that it could be built. A legend wants that when Grubenmann showed to the Councilmen the timber model of the bridge, they derided the design. He, in order to convince them, did not produce calculations demonstrating its feasibility, but stood up on the model (Blaser 1982). In that age in fact designers, and in particular carpenters, still based themselves on traditions and intuitions in dimensioning their structures. So Grubenmann was forced to elaborate a new design, in which the track was supported by the masonry existing pile at the center of the river bed. Such design was shown to Authorities in 1756, accompanied by a new timber model, a copy of which is reproduced in Figure 10. A drawing of this really built bridge is brought back by Krafft in his treatise (Krafft 1805), but it had already been published in Basel in 1803 in the work «Plan, Durckscknitt und Aufriss der drey merkwürdigsten höllzernen Brücken in der Schweiz» by Christian de Mechel, who had reproduced it in an etching of 1802. A previous table with the drawings of the Schaffhausen bridge was due to Christoph Jezeler, «Stadbaumeister» of Schaffhausen from 1766 and 1769 (Navone 2002). Rondelet, in the first edition of his treatise (Rondelet 1810), gives the description of Krafft, who erroneously dated the construction of the bridge to 1770–1771, and reproduces its drawing in Table 143.

Krafft and Rondelet both report that Grubenmann had designed a bridge with only one span, but was forced to make it rest on the existing central pier. They affirm, but this is another legend, that, once construction was completed, the bridge did not rest on the central pier, but it balanced with a gap of 18 inches above the pier. Only after some years, when the relaxation of whole structure was completed, the bridge leant on the central pier. Such circumstance is rightly contested by Emy in his treatise:

> Si è preteso che Grubenmann, per dimostrare la potenza dell'arte sua, avesse costrutto questo ponte in guisa che non posava sulla pila di mezzo, e che i magistrati esigettero che vi si facesse poggiar sopra usandovi delle zeppe: ciò che troviamo poco probabile perciocché non essendo il ponte in linea retta, ma formando angolo e cadendo il centro di gravità fuori della linea che unisce gli assi delle due testate, lo si avrebbe esposto ad un movimento di torsione proveniente dal suo peso (Emy [1841] 1856).

Also the most famous survey of Schaffhausen bridge first design is due Christian de Mechel who, in 1802, reproduced it in a etching by plan, longitudinal and cross sections, identifying it erroneously with the design of a bridge on Limmat river at Wettingen, destroyed, like that one of

Schaffhausen, in 1799, by French troops. This error of attribution was repeated by Rondelet, who in his treatise, in the paragraph entitled Wettingen Bridge, reports integrally the description of the first design of

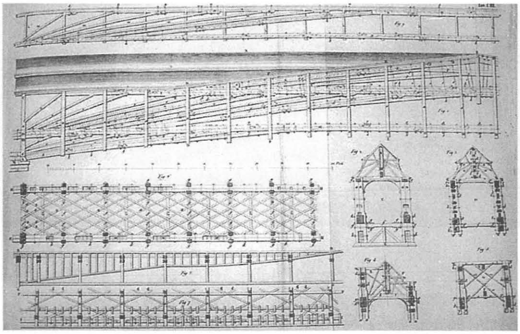

Figure 11
Plate 103 of sixth edition of Rondelet's treatise (Rondelet [1810] 1833)

Schaffhausen Bridge, given by Christian de Mechel, and reproduces it in Plate 103, (Rondelet [1810] 1833), Figure 11.

The same error is repeated by Emy ([1841] 1856) who reports its description with reference to his Table 134. In truth, the covered bridge really realized at Wettingen by Grubenmann brothers is that one with an arch structure brought back in Figure 9, destroyed in 1799. The contract for its construction was stipulated between Abbot Caspar Burgisser of the Cistercians Wettingen Abbey and Hans Ulrich Grubenmann in 1764. To its construction, which lasted from 1765 to the end of 1766 took part also Johannes Grubenman and two sons (Kottmann 1958). In the same site, in 1818–1820, was built a new bridge with two spans of 36 and 19 m, resting on a masonry central pier; the larger span, which is a covered one, is still existing.

Description of the structure

The description of Schaffhausen bridge first design, which has been taken as reference, is that one of Christian de Mechel, brought back by Rondelet ([1810] 1833); the survey has been Table 103, Figure 11.

The bridge has one free span of 118,80 m, with track clean width of 5,00 m. The inner height, under the covering structure intrados is of 5,50 m. The covering, with a variable profile, is of a mansard-roof type. In analogy with the existing documentation of other contemporary covered bridges, it has been assumed that the cover mantle was realized with wood tiles and that the sidewalls were completely closed by timber tables. Moreover the presence of five openings for each side has been considered, illuminating the middle of the bridge. In conclusion the building aspect could have been like that shown in Figure 12.

This prospect shows the exceptional slenderness of this construction, which can be synthetically expressed by the ratio between the height at middle point and the length of the span, and is equal to 1/11, in contrast with that one of some contemporary covered bridges with values variable between 1/3 and 1/7.

The designed structure is contained in the vertical surfaces aside the track and is of composite type, formed by trusses with inclined struts, disposed above and below the deck. All the structural elements in vertical plans are fixed by sturdy double hanging posts, at a mutual distance of approximately 5,20 m. These posts embrace and support the track main beam of cross-sectional dimensions 0,26 × 0,95 m, the covering structure impost beam, of maximum cross dimensions 0,60 3 1,45 m, the rafter, of 0,50 × 0.55 m, and all the inclined struts, of variable cross-sectional dimensions from a minimum of 0,26 × 0,26 m to a maximum of 0,70 × 0,80 m.

About connections in these structural elements, Christian de Mechel says:

> Le grandi travi . . . ed i grandi puntoni . . . sono formati . . . da molti pezzi innestati alle loro estremità e commessi a denti nella loro lunghezza, serrati l'uno contro l'altro da cunei, e legati insieme con ferri a vite e dadi (Rondelet [1810] 1833).

The double hanging posts are constituted by couple of symmetrical elements, regarding the vertical plan, at a mutual distance of 0,30 m and of variable cross-sectional dimensions along the height: in the lower part they are 0,60 × 0,38 m, in the intermediate part 0,45 × 0,38 m, and in the upper part 0,30 × 0,38 m.

The covering structure is articulated on a central backbone, contained in the bridge symmetrical plan, constituted by a lower longitudinal beam of constant cross-sectional dimensions of 0,34 × 0,60 m, and an upper one of variable cross dimensions from 0,30 × 0,28 m to 0,85 × 0,28 m, connected by double hanging posts composed by two parallel elements of 0,28 × 0,14 m, placed at the same mutual distance of

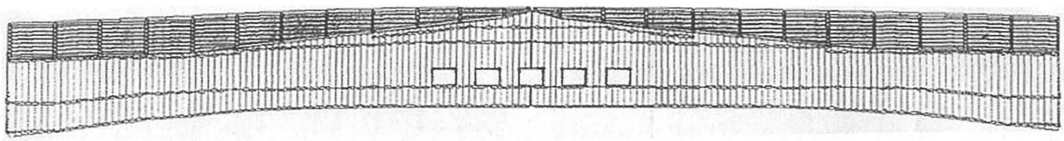

Figure 12
Reconstruction of the prospect of Schaffhausen Bridge first design

the main double hanging posts. In the vertical surface between upper and lower beams numerous inclined struts are arranged, with a cross section of $0{,}26 \times 0{,}26$ m. From the central backbone many inclined joists depart which, with variable cross-section and inclination, support the covering structure conforming timber elements. Such joists, together with the beams of cross-sectional connection, re-united at the top of the double hanging posts using metallic aids, make the covering structure a solidly jointed part of the bridge main structure.

The presence of a backbone is characteristic of the most important carpentry works of the Grubenmann family, already experimented in building churches coverings, like in the Evangelic Church of Grub, where the roof trusses, placed at very small intervals, form a single spatial structure because of the presence of a longitudinal polygonal skeleton (Killer 1988).

The secondary structure of the bridge deck is constituted by connecting crosspieces of cross-sectional dimensions $0{,}35 \times 0{,}45$ m, rigidly jointed to the double hanging posts also with iron strips and bolts.

The combination of all the described elements, defines a spatial entity with a box-like behavior, which as been understood by Christian de Mechel who tries a static interpretation based on the mutual support of the main elements of the structure (Rondelet [1810] 1833).

Such box-like behavior is exalted by the presence of the timber sidewalls coverings, and of course is subordinate to the hypothesis that the covering structure is rigidly jointed to the side one, and can be taught as a part of the resistant scheme. This circumstance is realized when the timber elements in the horizontal plane at the level of covering impost form a quite indeformable frame.

STRUCTURAL VERIFICATION

The spatial structural scheme of Schaffhausen Bridge, based on the drawings of Christian de Mechel, is constituted by one-dimensional elements with joints reproducing the described connections. The three-dimensional sight of the reconstructed scheme is brought up in Figure 13 where the outer cover is only partially reproduced in order to allow viewing the structure devised by Grubenmann.

The analysis has been led in linear elastic range.[1]

Enforced laws oblige to take into account, in bridge dimensioning, the possible presence of very large accidental loads which, therefore, constitute one remarkable share of total loads to be computed. For eighteenth-century timber bridges, built in Swiss valleys, it can be thought that the most important loads are dead ones, especially in presence of structures with an extremely closely-traced texture of beams with large cross-sectional dimensions, and also of timber coverings of sidewalls and topping.

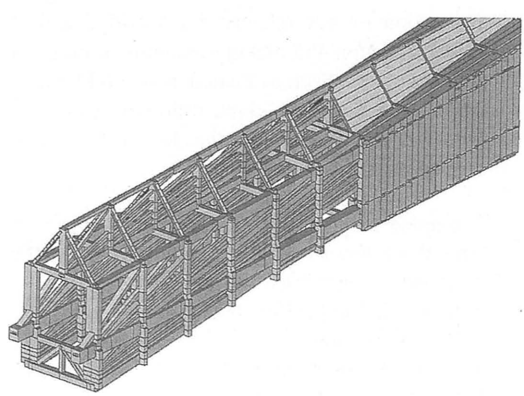

Figure 13
Rendering of the devised structural scheme

Knowing the employed wood essence is therefore determinant at the aim of right appraisal of dead-loads. Blaser (1982) says that the most used essences were oak and fir, as larch, technically more favorable, would have turned out excessive expensive. In lack of indications about the essences which Grubenmann meant to use in building his first designed bridge, and being based on the circumstance that the second bridge was built in fir, the employment of this essence has been assumed, with a density of 450 kg/m^2. In making calculations a value of 500 kg/m^2 has been adopted in order to take account of the great amount of iron fittings forecasted to strengthen the joints.

The overload on the track has been deduced by Rondelet's considerations about calculation of bridges assigned to heavy coaches passing:

> Supponendo il ponte destinato al passaggio di grosse vetture, il maggior peso che possa aver da portare la parte di mezzo, prendendo 6 piedi per lo spazio fra ciascuna armatura, non potrebbe essere più di 20.000 libbre. Questo carico equivale per ciascun armatura allo sforzo di un peso di 10.000 libbre situato sul mezzo (Rondelet [1810] 1833).

An overload of 2,35 kN/m^2 on the whole track length can therefore be deduced, even if this value is lower than actual standards.

The evaluation of overloads on covering structures due to snow effect has been carried out in compliance with the enforced Italian laws, using a referring value of q_{sk} = 0,90 kN/m^2 brought back in Euro Code 1 (2–3).[2]

Wind action is very relevant for a bridge with a closed cross section and making assignment on Euro Code 1 (2–4),[3] it as been evaluated as q_s = 6 kN/m^2.

Numerical analysis has given following values of maximum normal stress in the leeward vertical surface, which is the most stressed:

- in upper longitudinal beam, near the third inclined strut upper end $\sigma \sim 17$ MPa
- in lower longitudinal beam, in the maximum positive bending moment section $\sigma \sim 13$ MPa
- in the third inclined strut $\sigma \sim 19$ MPa

Corresponding diagrams of normal stress and bending moment are reported in Figure 14.

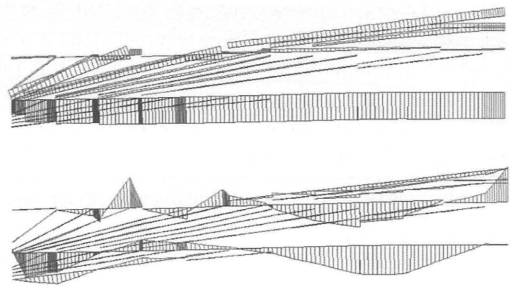

Figure 14
Normal stress and bending moment diagrams for the leeward surface

In the symmetry surface, maximum stress values are:

- in upper longitudinal beam, near the last inclined strut upper end $\sigma \sim 19$ MPa;
- in lower longitudinal beam, in the maximum positive bending moment section $\sigma \sim 13$ MPa;
- in the last inclined strut $\sigma \sim 16$ MPa.

Corresponding diagrams of normal stress and bending moment are reported in Figure 15.

A verification made using maximum allowable stress criterion shows that obtained values of normal stress are larger than the admissible value in bending parallel to longitudinal fiber, which is 11,00 MPa. Nonetheless with reference to known values of rupture for fir wood with a density of 450 kg/m^2, which waver about 70 MPa (Giordano 1999), it can be concluded that bridge structure,

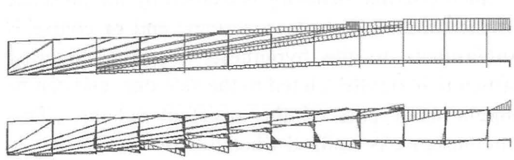

Figure 15
Normal stress and bending moment diagrams for the symmetry surface

even if strongly stressed, is very far from local collapse. Moreover maximum inflection measured in middle section is about 0,69 m, which corresponds to a ratio of 1/170 to the span, and therefore only little above the admissible value of 0,59 m, corresponding to 1/200.

Concluding remarks

Structural analysis shows that the first solution proposed by Hans Ulrich Grubenmann to build Schaffhausen Bridge, is a quite right one. In fact, safety conception about timber structures, as developed in the second half of twentieth-century, was not even guessed, at Grubenmann's times, by people operating in building field, especially for what concerns aleatoriety of rupture limit strength of wood. This lack of notions was sometimes overcome by great experience in choosing timber logs more secure in relation to their specific structural arrangement.

So the high exercise normal stress values measured wouldn't compromise the full utilization of the bridge. Also Christian von Mechel reports, in his just quoted writing of 1803, that evaluation of wood strength, must be commensurate to its weight, as he says could be deduced by Busson's works, exposed to Paris Royal Science Society in 1739 (Rondelet [1810] 1833).

The most interesting aspect of the studied structure is due to its spatial behavior, surely guessed by Grubenmann. In fact he brings about a large stiffness increase inserting a truss along bridge axis, which supports the covering system, and is rigidly connected to side surfaces structures, and also making the timber coverings of side surfaces and top, as well as the cross structures of the track, have cooperating structural parts. This behavior has been verified confronting structural analysis results of bridge spatial schemes with and without timber coverings on side surfaces and on the top. The presence of the timber shell causes a maximum stress reduction of about 15% in side frames and about 40% in central backbone structures. Also maximum inflection value shows a considerable reduction of about 30%.

Moreover it is interesting to observe that the arrangement of timber structural elements in side walls, characterized by a gathering of inclined struts near the banks with their inclination growing from middle axis towards side leanings, induces in side frames the arch behavior. In fact inclined struts and upper beams at the covering impost form an arch which takes compressive stresses due to superimposed loads and brings them at the ends of the lower beam, which behaves like a true tie-beam, supporting the considerable pull. As the arch is quite flat, bending effects are preeminent in all structural elements.

Even if it has never been built, this bridge represents an extremely dared structure and testifies the high level of Swiss carpentry art at the end of eighteen-century. It is significant the great admiration for this bridge shown by treatise writers of nineteen-century: Rondelet ed Emy, who both believed it was a really built bridge at Wettingen, report it as one of the most significant example of timber carpentry and mourn over its destruction with passionate tones.

Notes

1. The software used is Nolian program by Softing.
2. *Basi di calcolo ed azioni sulle strutture*, 2–3 Azioni sulle strutture, carichi da neve. UNI ENV 1991. 2–3 October 1996.
3. *Basi di calcolo ed azioni sulle strutture*. Parte 2–4. Azioni sulle strutture, azione del vento. UNI ENV 1991. 2–4 March 1997.

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Pietro da Cortona's domes between new experimentations and construction knowledge

Annarosa Cerutti Fusco
Marcello Villanni

During his lifelong activity Cortona has been dealing, in several circumstances and in many projects, with the problem of the construction of domes. We will examine only the really built structures, making a distinction between remodelling old domes, designed by others, and new works (Cerutti Fusco.Villani 2002).

Among the Cortona's domes, first we will mention the intervention in S. Maria in Vallicella (1647–1651) and S. Maria della Pace (1656–1659); then, the church of Ss. Luca e Martina (begun in 1634, but continued in a long process of construction), concluding with Cortona's masterpiece of the end of his career, the remarkable dome of the already half built Ss. Ambrogio e Carlo al Corso (1668).

The whole matter, related with the domes'construction in its cultural context, can be investigated either from the side of theory, or from the side of practice.

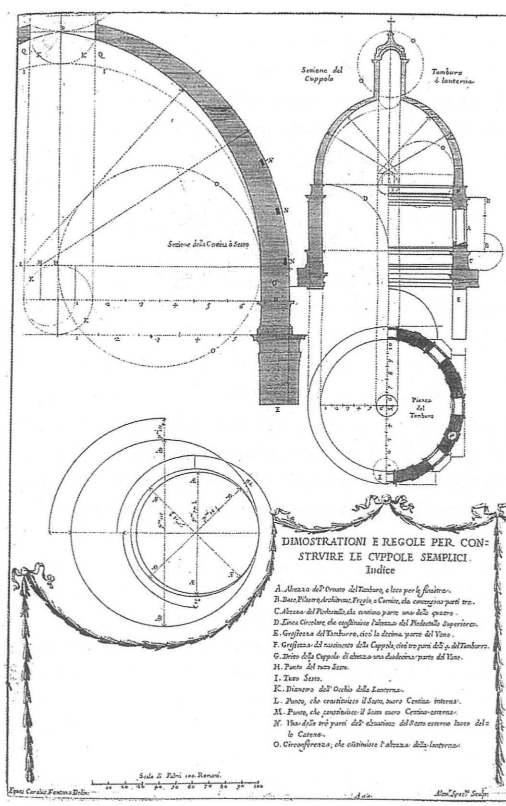

Figure 1
Carlo Fontana's rules for dome's designing (from *Regole per le Cupole Semplici)*

The theory

As far as the seicento theory on mechanics and static stability is concerned, it is possible to say that the criteria of domes project were dominated by traditional principles of design. Geometry and proportion, plus some basic recommendations on correct and useful construction procedure were later on pointed out as well by sound architects, such as Carlo Fontana, Guarino Guarini and Bernardo

Antonio Vittone. From this point of view, the most important texts in baroque Rome can be considered Fontana's *Dichiaratione dell'operato nella cupola di Monte Fiascone colla difesa della censura* (1673) and *Regole per le Cupole Semplici*, included in his famous *Templum Vaticanum* (1694). According to Fontana, the design of the various elements of the domes descends first of all from simple geometrical principles, and great importance is attributed to the comparison among the most celebrated roman domes. Only a few notes are reserved to the different properties of the building materials.

However, prevously, we must here mention Teofilo Gallaccini's treatise *Sopra gli errori degli architetti* (about 1625), at that time still manuscript, but known in the Barberini circle, in which Cortona was well introduced. Physician and lecturer in mathematics in Siena, Teofilo was willing to spread a culture on erecting a sound perennial structure, a culture he acquired studying the roman ruins and the fortification, an interest that he shared with Galileo, whom he was acquainted with. *Sopra gli errori . . .* concerned the construction knowledge, a complex set of rules for building correctly, according with mechanics, including hydraulic, the analysis of soil and foundations, the building procedures, the properties of materials, the technique of masonry. Addressing himself more to the «ministers» or to the patrons than to the architects, whom he was inclined to distrust, he covered a lack of writings, proposing a simple scientific method to design and to control the proportion of the structures, to preview and to diagnose local failures and possible errors or abuses. Indeed his treatise, presenting architecture more as a science than as an art, was at that time the most advanced bridge between practice and theory.

For many reasons, also related to his famous controversy with the Inquisition, Galilei's influence was wide and perceived also by patrons and architects: for example, an important Galileo's contribution to the current knowledge about the equilibrium of the vaults was not only the rigorously mathematical method applied to the problem, but also the bringing in the concepts of friction and boundary conditions (Benvenuto 1981, 102 ff.; Di Pasquale 1995–1997). However, this new approach was particularly useful when the diameter of the circle/oval, or the width of the polygon (usually an octagon) at the base of the dome reached the critical measure of about forty meters or more, like the diameter of the Pantheon, or those of St. Peter's and S. Maria del Fiore's. A diameter of around twenty and thirty meters was already considered significant, so that one should assess carefully the vulnerability of the structure, and evaluate the risk of cracks.

The average diameter of the roman baroque domes is usually bound between twelve and twenty meters —the cupola del Gesù, one of the largest, measures about 80 palmi (17,87 m)—, while the height of the dome, including the lantern, ranges between fifty and sixty meters, or a bit more. In these cases the construction technique could easily follow the traditional path, although many problems challenged the erection of a dome of medium size. Those included the choice of proper and sound materials, the process of construction down to the right moment in which safely the inner timber centers should be freed, the high costs to afford, how to finance the enterprise, how to handle the question of a longlasting *firmitas*, applying some progress of the new science in order to achieve unexpected results in the work of art (Marconi, D'Amelio, De Feo in Conforti 1997). So, in spite of its improvements, Galileo's new mechanic was not substantially assimilated by the roman architects, even by those brilliant experts in mathematics, like the jesuit Orazio Grassi, the author of the never built dome of S. Ignazio.

In the sixteenth and seventeenth centuries the architect usually supposed the homogeneity of the material, conceived as not elastic, but rigid, and considered most of all the proper weight of the material, so that the thrust could be almost exclusively compressive. For example, the study of the lateral tensions, or the behaviour of the pendentives, spherical triangles which act as a transition between a circular dome and a square base on which the dome leans, were usually omitted. Iron, stone or even wooden rings were built in the orizzontal layers of the dome in order to contrast the lateral tensions.

The diameter of the dome, as we have underlined, was the main reference measure. The treatises of architecture generally avoided to explain in to details the practical side of the architecture on two main grounds: first because in the building-yards there were competent masters and skilled workmen that knew well by tradition their «techne», including the employment of innovative machines, mainly through

direct experience and oral information, second because the evolution from the medieval and renaissance heritage was extremely slow and for domical vaults of medium dimensions the traditional knowledge was adequate enough.

The question of the light

On the subject of the baroque theory on the domes' design we have to mention a lively debate on the way of lighting, which took place inside the Accademia di S. Luca, where Cortona, Principe in the period 1634–1637, had been teaching painting during his all life. There were two different positions: the first recommended the way of lighting in the manner of the ancients, through a round open eye cut-off at the apex of the dome masonry, as in the Pantheon, so that the light, falling from the top, could directly flow inside with changing rays' inclination; the second, supported by Cortona, maintained the manner of the moderns, through the lantern, with an indirect and diffused illumination. This fruitful discussion involved a broader controversy, originated in the field of the music, early in Florence with the Camerata fiorentina and Vincenzo Galilei, and spread in the field of the arts between the supremacy of the ancients over the moderns or viceversa. Pietro da Cortona and the roman baroque architects all applied the modern way for illuminating their built domes, while attempts to propose in some designs the Pantheon like system were not generally put in execution: in Milano the debate concerned the reconstruction of San Lorenzo Maggiore's collapsed dome, rebuilt in 1619 by Martino Bassi, and in Florence such a system was adopted for the Cappellone dei Principi in San Lorenzo.

The debate about the light —a dominant theme of the period, rich of theological and metaphysical meanings— was particularly worked out in connection with painting. The subject had been already treated, in particular way, by Leonardo in his theory of light, shadow and penumbra, and by Serlio in dealing with the transparent light. In Seicento, the new science and theory of optic and perspective contributed to develop this argument, which had received a particular attention on the wave of the revolutionary works of Tintoretto, Caravaggio and the new scenic art related with the melodramma. Borrowing from the practice of painting the concept of secondary light, also Galileo focused his attention on the problem of illuminating, that later was analytically treated by A. Kircher, in his influential work *Ars magna lucis et umbrae* (1646). Once again, Galileo was leading the path in the theoretical field, with evident consequences in isolated works of art, as in the renowned case of Cigoli's *Madonna dell'Immacolata Concezione* in the Cappella Paolina (1610–1612), in which the lunar globe, in his increasing phase, appeared for the first time with the shadows and shapes observed by Galileo. The science of optics produced a new technology in the . . . observation of the nature, so that the artist was stimulated to make incursions on unexplored fields. New horizons opened up to the celestial observation through the telescope, while through the microscope it was much easier to investigate the marvellous details of the small living creatures.

Not only there was an influence of the science upon the art, but also viceversa, and especially the art of representation, the theatre, was essential in shaping the mentality of the time. Two were the main tools, aside from the music, that could persuade and stir an emotional response in the spectator: light and perspective, in motion. N. Sabbattini, *Pratica di fabricar Scene..* (1637,1638) illustrated thoroughly the deceptive plays of the perspective of infinite space in the stage, deeper enough to allow sometimes a double backdrop of the scene and the sophisticated tools, in order to manipulate artificially light and darkness (usually through oil lamps, candles or torches, and occasionally with the help of depicted glass in different shape, sometimes full of water, as Serlio already stated) in the theater, both in the auditorium and in the scene, preferably from hidden sources, as described also by J. Furttenbach the elder, *Mannhafter Kunst-Spiegel* (1663).

The interest of the architects of the time about illumination of the domes, and the seminal role attributed to the light and music in the post-tridentine churches appear evident, already with the cupola del Gesù and the connected process of design. The domes were the ideal theatre, a space of representation, where the eyes and ears of the spectators were caught in profound emotion and persuaded with celestial and spiritual experiences.

All these aspects should be considered in relationship to the specific architectural culture of the

time, remembering that Berrettini tried always to apply the three main vitruvian categories: *firmitas, utilitas* and *venustas*, among which often conflicting tensions could easily rise. Evaluating the extraordinary bright experimental and technical solutions adopted either by Borromini or by Cortona for building the vaults and the domes, while Bernini preferred more traditional systems, in baroque Rome we can distinguish two main streams in the practice of architecture, including the building systems, the first based on the lombardo-ticinese tradition, the second on the tuscan one, to which belonged Cortona, who however was aware of the more advanced experimentation of the ticinese school.

About Cortona's domes, we select now two matters strictly interwoven: the adaptations and refurbishing of previous existing building, and finally the new constructions.

The intervention of Cortona on previous built domes

In the complex field of new qualification of pre-existent domes, Berrettini worked at the domes of S. Maria in Vallicella, between 1647 and 1651, and of the church of the agostinians, S. Maria della Pace, redecorated between 1656 and 1659. As far as it concerns the first intervention, designed to give light to the fresco decoration, commissioned to Berrettini, Pietro opened a range of oval windows at the bottom of the shell.

This solution changed the effect and the perception of the central space, enlarged by the glasses' transparency. The improved dome, wholly superbly depicted, became more graceful, floating on the light as if it were suspended in a void and, as sacred image, acquired a new emphasis, through a continuing spirited movement of light and shadows, and a dynamical *chiaroscuro*.

> Indeed the interruption of the continuity of the material in an extremely weak peripheral area at the spring of the dome caused, within twenty years, local failure generated by overstress in the structure, so that Carlo Fontana had to study some devices, in order to solve the critical situation (Hager 1973).

For S. Maria della Pace's octagonal tiburio-cupola, Cortona chose to evoke the Pantheon, at which he was referring either in the short half oval portico in the facade, and in the aspect of the stepped outer shell, designed in the manner of the ancients (*a gradoni*).

To the remodelling of the exterior in this fashion not rare in Rome (we can mention, for example, S. Maria Scala Coeli by Giacomo della Porta), corresponds the redecoration of the interior, with stuccoed not structural ribs and octagonal coffers, with the heraldic symbols of Chigi and Della Rovere's noble families, patrons of the church.

The systematic adoption of metamorphic stucco in the decoration of the ceilings was characteristic of

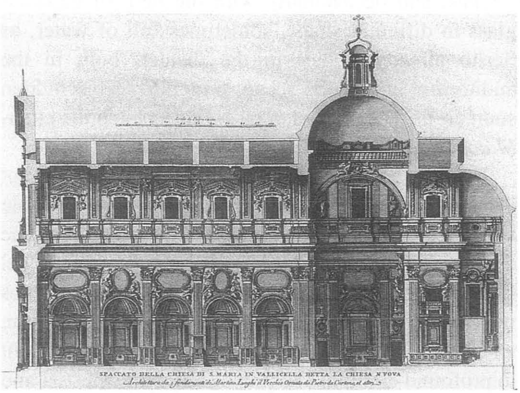

Figure 2
Rome, S. Maria in Vallicella: longitudinal section (from D. De Rossi, *Studio di Architettura civile* 1702–1721)

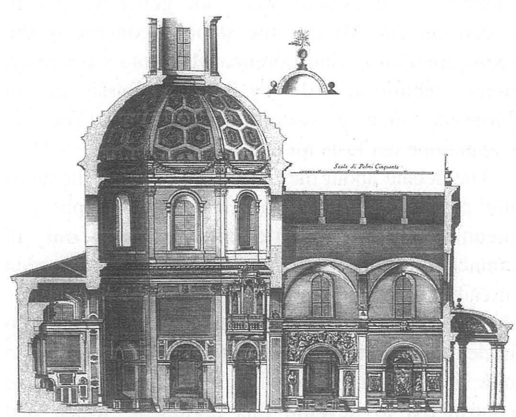

Figure 3
Rome, S. Maria della Pace: longitudinal section (D. De Rossi, Studio d' Architettura Civile, v III, Rome 1721)

baroque architects, who applied it in original experiments and solutions, in new buildings or in refurbishing old ones, with a rather low cost and prompt execution. Often the artists had the intention of creating illusory unforeseen effects, fictive surfaces and depths in space, imitations of artificial, fantastic or natural elements as clouds, or materials from the hard marbles or precious metals like gold, silver and bronze, to the soft tissues like curtains and even false objects. The need of a longlasting results implemented the study and application of various different methods to make a robust resistant stucco, through a choice of different constitutive elements. The yard-art of stucco developed in particular in the lombardo-ticinese tradition, but the masterpieces are connected with Bramante, Giovanni da Udine, Raffaello, Giulio Romano, Peruzzi, Cataneo, Sansovino, who gave a synthesis of the Lombardo-Veneto and Tuscan culture, till Pirro Ligorio, Aleotti and Maderno. Cortona, well aware of the new trend in the stucco-forte or marble-stucco (*scagliola*) decoration as a kind or substitution of sculpture, as did Bernini and Borromini, applied the stucco as a complementary part of his dome architecture, in order to create movement of elements in the interior drum and shell, inaugurating a more plastic conception of the geometry of the dome and in the meantime hiding the source of light in the attic, an effect that he achieved perfectly in the interior of Ss. Ambrogio e Carlo al Corso.

It is interesting to mention, among many chapels built by Cortona, the interior decoration of the vault of the cappellina of S. Filippo Neri in S. Maria in Vallicella. In this tiny chapel, executed according to a design of Cesare Guerra, for the first time Pietro Berrettini inaugurates a new style of ornamentation in the interior of a dome: the idea was to create an attractive contrast between ribs and coffered ceiling, using with sophistication the art of the stucco.

For its consequence on the Cortona's knowledge about the difficulties of domes'construction, we want finally mention the discussion about the way Borromini, who was later without much respect dismissed, handled the completion of the lanternino over the cupola of S. Agnese in Agone, mausoleum of the Pamphilj's family in Piazza Navona. As the documents reveal, Borromini didn't dismantle the armours (timber centers) because of some cracks that had already appeared in the structure, and waited too long time without giving explanation about the solution of the problem. Since we cannot go in to details, we mention only that Cortona was also summoned to give advice: The decision of a new commission was to lighten the structure of the dome, outwardly giving up the travertino in the drum, and internally the marble of the entablature, plus to postpone to free the dome from the armours, in order to compleat all the masonry structure of the church at various levels, including the facade in the rear, on Via dell'Anima.

Cortona's new domes

We study now the domes realized by Berrettini *ex-novo*: in the first place, the dome of the church of Ss. Luca and Martina, started to be erected in 1634 and in process up to 1669, year of the death of the Cortona, and later finished by Ciro Ferri, the most faithful pupil of the Tuscan. In the second place, we investigate the dome of Ss. Ambrogio e Carlo al Corso, although we must specify that the pillars of the church of the Nazione Lombarda were already partly built. The dome was finally commissioned in 1668 and has been entirely designed by Cortona, although realized mostly after his death. The two domes, crowned with a lantern, are medium size (diameter around 14 m), and have a single shell, with a vertical oval curve profile. The shell, marked with external ribbing, is raised above an attic and a drum with clerestory windows. The materials employed in the masonry domes are travertino, a local porous calcareous stone, peperino, a harder stone, bricks, tevolozze, broken reused old tiles and with limemortar, in order to lighten the load of the structure. The domes, externally covered with layers of lead, in the peculiar roman tradition, are provided with encircling ties at the periphery, in order to counteract the lateral thrust and to prevent spreading.

Ss. Luca e Martina

About the dome of Ss. Luca e Martina, the church of the Pontificia Romana Accademia di S. Luca, in the Foro Romano, we don't recall the single phases of construction (Noehles 1970; Cerutti, Villani 2002, 68 ff.; 195 ff.). About the dome of the Nazione

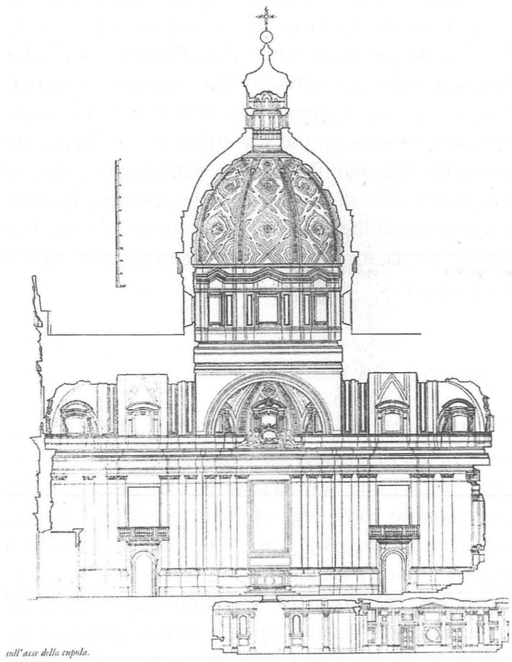

Figure 4
Rome, Ss. Luca e Martina: longitudinal section (from Noehles 1970)

Figure 5
Rome, Ss. Luca e Martina: view from the Capitol hill

Lombarda church, dome built according to Berrettini's design, in about ten years after his death, we will give a brief account of the construction history, together with a description of the materials used and the specific technique of the structure.

In both cases we should notice that Cortona in his domes carefully studied the mutual relationships between the interior and the exterior shells: Pietro set up a sometime ambiguous interplay between ornament and structure, between dome and interior space of the church, between dome, urban and territorial significant interaction and view.

In alternative with the plastered flatness of the interior, predisposed, as in many cases in baroque Rome (from S. Maria in Vallicella to S. Agnese in Agone), for the fresco decoration, in his new built domes Cortona, even against the will expressed by many academicians in the case of Ss. Luca e Martina, refused the painted ceilings, merging instead two different traditional ways of vaults ornaments: the first way was through coffers (*all'antica*), with the perspective effect of diminishing the geometrical pattern according to the form of the inner shell. Inspired from the example of the Pantheon, the coffers system had been already applied on the dome of the Sacrestia vecchia of S. Lorenzo, in Florence, by the «divine» Michelangelo, whose great authority was widely acknowledged, especially by Pietro. The second way was the internal plastic ribs, of long-standing tradition (again Michelangelo, or Antonio da Sangallo and Scamozzi, for example). In fact there had been already realized the combination of frescoes and internal ribs, which, provided that were not part of the structure, as in the Cupola del Gesù, sometimes have been even erased after the construction, to allow a full space for painting. The coffers and the ribs had

not yet been combined together until the dome of S. Filippo Neri. As authoritative scholars like Noehles (1970) and Benedetti (1980) have shown, the patterns of coffers represented by Cortona in his domes, like those of Borromini, were dense of Christological meanings, and exerted an enormous influence on countless works of that kind (documented since the first Concorsi Clementini in the Accademia di S. Luca). This new system introduced in the Chapel of S. Filippo Neri and later adopted systematically by Cortona was bound to become a paradigm, starting with Ss. Luca e Martina. A similar mixed system has been applied also by Bernini. He employed coffers shaped in geometrical forms, the octagon, with heraldic symbols, in the ceiling of S. Andrea al Quirinale, of S. Tommaso di Villanova in Castelgandolfo, and of S.Maria Assunta in Ariccia. Evidently Bernini was influenced by the taste of Alessandro VII, who intended to stress the heraldic arm of his family, as we can see as well in the interior dome of S. Maria della Pace. The choice of strictly geometrical shapes shows also that Gian Lorenzo inclined to expressions more in keeping with a taste *all'antica*, based on the model of the interior of Pantheon, for which he had studied a refurbishing wit Chigi's armorial bearings. The same classical taste, probably suggested also by Pope Alexander VII, occurred for the external shape of the outer shell: both in S. Tommaso da Villanova and in S. Maria Assunta in Ariccia, built for the Chigi's family, the ribs were stressed in a less plastic and original way as in Cortonas works. Berrettini played with the not structural inner ribs overimposing different planes of ornaments and figures, so to suggest deceptively a deeper thickness and a higher cupola.

As far as the function was concerned, the way in which the light (*i lumi*) entered inside the dome depended also if the ceiling was decorated with a fresco, or with a sculptured either technical or purely ornamental system, such the inner ribs, more or less decorated. Cortona designed a coffered ceiling, in a mixed arrangement, made with stuccoed ornamentation, not only geometrical, but also symbolic, carved in plastic and meaningful complex. The manipulation of the light, a constant concern shared also by Bernini and Borromini, answered in fact to the specific function and significance of the space: in the case of the dome of Ss. Luca e Martina, Pietro chose to convey light from many sources, diffused through various vertical levels. From the top we observe a very diaphanous lantern (more exactly one could say a cupola, in italian cupolino) enough large and designed in a complex exotic, apparently bizarre, way (sometimes defined as bulbous cupola), the attic, in a way that from inside the dome the sources of light are not perceived, because are located so high or behind other elements, that are concealed or masked. As result in the interior the spectator could see, over the entablature of the inner drum, an additional flow of light. Cortona decided that the tall drum had to raise up completely out of the roof of the main nave, and that the dome should be well visible and evident, in relation to the façade: the question and the solution were not at all new, since they had been already discussed for S. Peter. Later the problem had been treated for the completion of S. Ignazio, around 1645, by Orazio Grassi, under the suggestion of many famous architects. In that occasion the jesuit father recommended to adopt a particular solution, as a consequence of a variation from the proportions established in wooden model and in the approved paper design of the Church: since the height of the prospect and side walls of the building had been increased, a taller drum was needed, to prevent the dome to disappear to the sight (Bösel 1985, pp. 160 ss). This choice of a high drum was bound to produce an effect that Bernini did not approved from the esthetical point of view: the spectator had to bend and overturn the head in order to observe the dome from the inside.

In Ss. Luca and Martina a tiers of windows originally pierced the attic, but this solution caused cracks and a local pathology (called peli), so that the continuity of the wall's masonry had to be restored. This drawback had already been revealed in San Carlo ai Catinari, notwithstanding the bricks' arches of unloading (on the exemple of the Pantheon), made in the structure order to convey the thrust in the correct direction, over the supports.

The solution of limited flows of light coming from the attic was then applied successfully in Ss. Ambrogio e Carlo al Corso. Inside, the sources of the light were concealed, thanks to perspective reasons: on one hand the high drum obstructed the view, on the other hand the strong interior articulation of the columns, together with the projecting members of the order prevented the sight to distinguish the windows of the attic, so that the illumination of the cupola

deceptively produced the effect of a kind of dissolution or dematerialization of the space, that contrasted with the plastic allusive protrusion of the external ribs in the interior.

Particularly interesting was the debate about the resistance of the two already built pillars, empied with lumachae stairs, that preceded the construction of the dome of Ss. Ambrogio e Carlo al Corso: it is clear how Pietro da Cortona, consulted as the most expert of his time about the «Theorica» (theory) of the domes, founded his conclusions on a rational analysis of seminal design criteria, both theoretical and pratical, useful for building cupolas. The construction knowledge about the domes, in the first half of seventeenth century, has been in general underestimated, because it has been related from one side to the scarce production of specific technical

Figire 6
Rome, S. Andrea della Valle: dome

Figure 7
Rome, S. Agnese in Agone: exterior view, detail

literature, and on the other side to more advanced mechanical theories.

In Rome, except the double shell of S. Pietro, the domes with slender proportion and vertical shape, previous to S. Ss. Ambrogio e Carlo al Corso, were S. Giovanni dei Fiorentini (1612) by Carlo Maderno, S. Andrea della Valle by Carlo Maderno, compleated by Borromini in 1621, S. Carlo ai Catinari (1612–1620) by the barnabita Rosato Rosati, Sant'Agnese (1657) by Borromini and Carlo Rainaldi, with the advice of the best experts, among which Pietro da Cortona (whose most faithful pupil, Ciro Ferri, painted the dome's ceiling).

In his built domes Cortona refused to pierce, as in San Pietro, Sant'Andrea della Valle and San Carlo ai Catinari, the outer shell with little holes which, though useful for the circulation of the air, evidently visually interrupted the curved line od the ribs and the compactness of the webs, diminishing the elegance of the whole urban image. Finally we can mention, if nothing else on the base of its proximity to Ss. Ambrogio e Carlo, S. Rocco's dome (around 1650), by G. Antonio De Rossi. In this last dome which, although very poor in sculptured element, was noticeable only for the large windows carved in the slender drum, De Rossi simply summed up the previous experiences, without expressing a work of art.

Moreover we can add to the possible reference for Ss. Ambrogio e Carlo al Corso's dome two jesuit churches: first of all the Gesù, by Giacomo della Porta (the author of the more successful dome of Madonna dei Monti), interesting for the inside more

than the rather awkward and heavy outer shell. For its large width it was carefully studied, as prooved in the documents about S. Ambrogio e Carlo al Corso (D'Amelio 1997, 200–201).

In the second place we should recall the debate about the S. Ignazio in which Orazio Grassi played an important role. We cannot forget to mention the countless domes in Italy, most of all in Firenze, Milano (San Fedele most of all), Genova and Napoli. The south capital of the spanish vice-kindom in the Seicento was a meeting place of many crosscurrents, in which a particular relevance assumed the ecclesiastical architecture built by the Jesuit, Barnabite and Theatine, especially in Rome, although the local traditions, for example in Napoli the polychrome majolicated covers of the domes, were profoundly different from the tuscan, ticinese and roman. In Napoli we recall, as possible references, the works of F. Grimaldi, Fra Nuvolo and most of all Cosimo Fanzago, who was active also in Rome.

Ss. Ambrogio e Carlo al Corso

Designed probably in the summer of 1668, and realized during the following four years, Ss. Ambrogio e Carlo al Corso's dome can be considered as one of Pietro da Cortona's masterpieces, as well as his last great work (Boido et alii 1987; Villani 1997; Cerutti Fusco. Villani 2002).

The finishing works of the church (transept, presbytery, apse, deambulatory and dome) realized by the Main Confraternity of *Natione Lombarda* (Lombard community) began in 1665. By the end of 1668, they were completed, except the façade and the dome. Pietro is mentioned as the author of the project of the dome in an original document (September 1668). On the architect's death (16th May 1669), only the lower part of the drum had been built; however, the whole dome was realized according to Cortona's

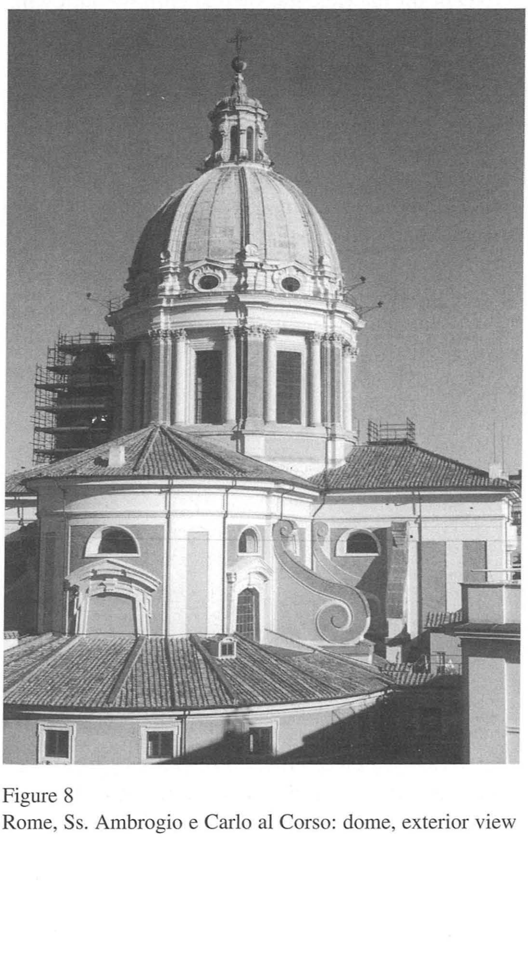

Figure 8
Rome, Ss. Ambrogio e Carlo al Corso: dome, exterior view

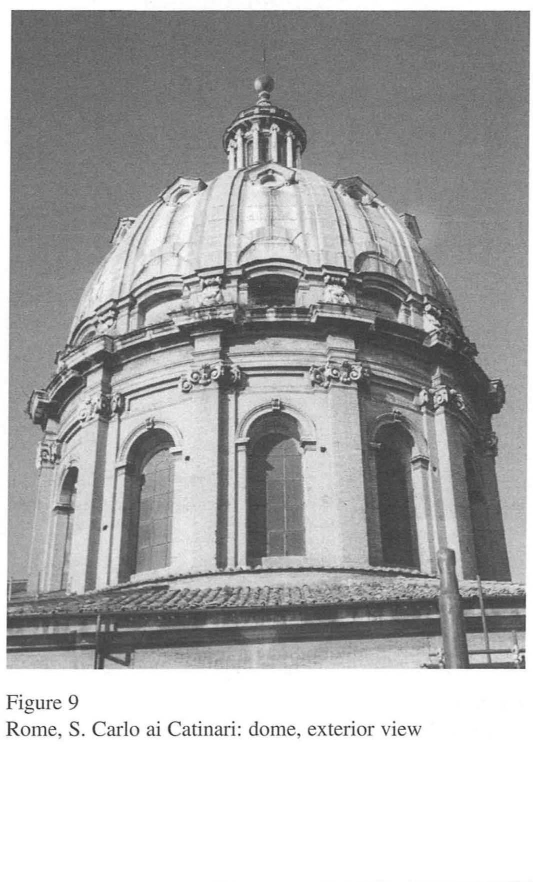

Figure 9
Rome, S. Carlo ai Catinari: dome, exterior view

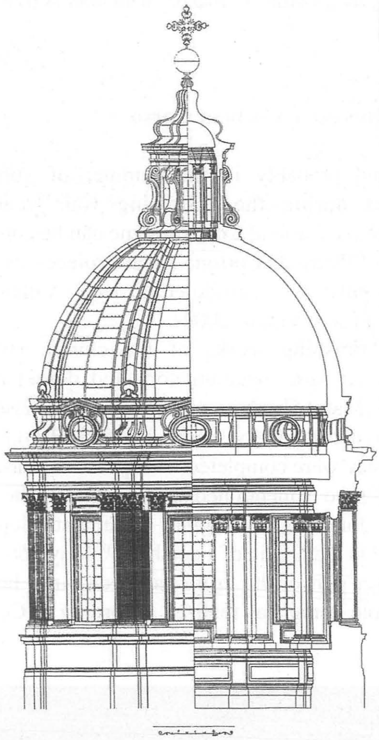

Figure 10
Rome, Ss. Ambrogio e Carlo al Corso: dome, front-section (from Boido, Mestrinaro, Tamburini 1987)

design, under the leadership of Tommaso Zanoli, the Confraternity's architect.

Starting from Rosato Rosati's S. Carlo ai Catinari (1612–1620), Pietro da Cortona designed such a «transparent» structure, with huge windows and no extended walls in the drum.

The adopted scheme was possible by concentrating most part of thrusts in eight cross-shaped brick pillars, with the little contribution of the contiguous columns (four for every pillar).

The arches which discharge the weight of the lantern and the shell to the pillars are hidden under the metal plating of the dome, unlike Rosati's dome, in which they are visible. Under the arches, between the drum and the shell, Cortona opened eight oval windows, increasing the inner lighting.

The presence of stone elements is extremely limited: a portion of the great basement of the drum, the inner and outer bases, and the capitals of columns and pillars.

Except for the ribs, the shell is entirely built with *tevolezza*: a mixture of incoherent bricks and lime-mortar, it can be regarded as a light, but sufficiently resistant material, the use of which was widely diffused in XVII century Rome (Bertoldi et alii 1983; Scavizzi 1983; Cirielli 1987). Of course, the stone is largely replaced by bricks and *tevolozza* to obtain a lighter structure. A great «cerchione di ferro» (iron ring), whose weight is about 1250 kilos,

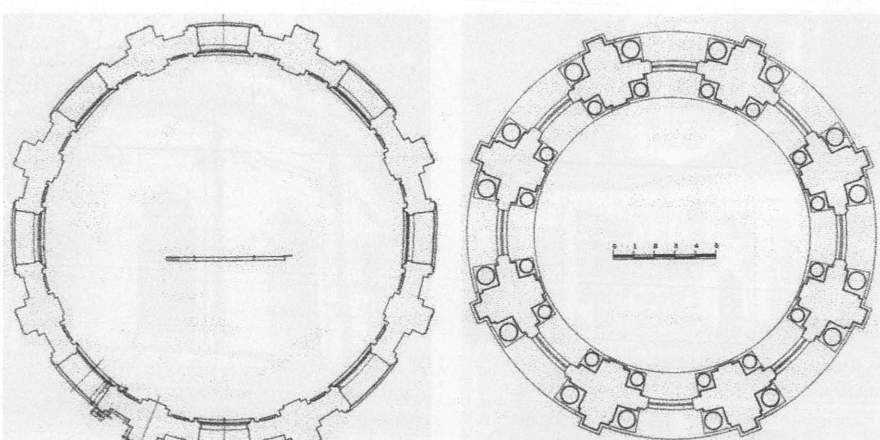

Figure 11
Rome, Ss. Ambrogio e Carlo al Corso: dome's plan, in comparison with Ss. Luca e Martina's one

Figure 12
Rome, Ss. Ambrogio e Carlo al Corso: dome, detail of the drum

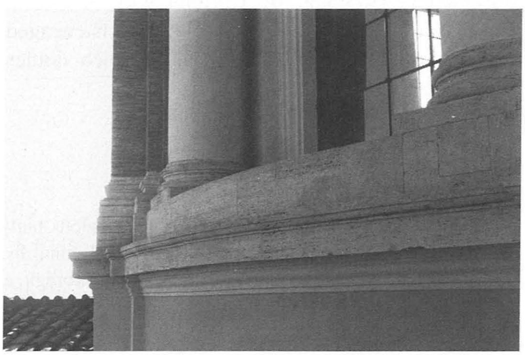

Figure 13
Rome, Ss. Ambrogio e Carlo al Corso: dome, detail of the drum

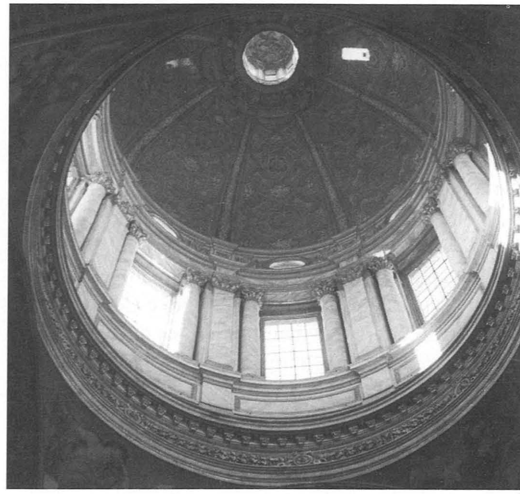

Figure 14
Rome, Ss. Ambrogio e Carlo al Corso: dome, interior view

is walled up, probably next to the impost of the dome.

The covering of the dome is made by lead slabs, according to the typical Roman tradition. Each piece —whose weight is about 5–6 Roman *libras* for *palmo quadrato* (cmq 500; every *libra* = 327g)— is fixed by lead nails.

Traditionally associated with the façade of S. Maria in Via Lata as an exemplification of Pietro da Cortona's late classicism, Ss. Ambrogio e Carlo's dome is a coherent step of the architect's evolution towards a more essential language. In this phase, the architectural order stands as the chief element in the composition; for example, the great windows of the drum, delimited by simple mouldings, present no decorations.

The main outcome of the lightening of the dome and the great enlargement of the windows of the drum is the impressive amount of the rays of light from outside into the inner space; from this point of view, the dome has no equals in Rome.

In this last masterpiece Cortona pushed the construction technique to its limits, when he made thinner the webs, in order to shape the exterior surface through a plastic multiplication of ribs, whose section instead of flat is unusually rounded, according to a hierarchy (Cerutti, Villani 2002, 119 ff.; 311 ff.). Furthermore he gave greatest prominence to the design of the light modulation. For this scope, Berrettini reached the valuable result of erecting at the drum level, in contrast with the opacity of the outer shell, a diaphanous openwork structure that, thanks to the wide windows, was reduced to an evident skeleton, an impression enhanced by the attic with clerestory. The drum is only apparently thin, because the thickness of the compact cross section is balanced between outside and inside. The spectacle of the verticalized dome, completely isolated from the roof of the church and transparent to the light from one side to the other, was perceived as unusual and audacious. Due to the urban environnment at the time

of its construction, the dome dominated more from distance than from the vicinity of the church. It is a *Gesamtkunstwerk* of sculpured architecture, in which a perfect equilibrium is achieved by Cortona: his ideal architectural testament.

One of the biggest in Rome, Ss. Ambrogio e Carlo's dome stands out as the perfect conclusion of the roman baroque evolution of the architectural theme. Although admired and studied all over Europe —and especially within the Accademia di S. Luca— owing to its originality, Ss. Ambrogio e Carlo's dome will remain with no evident imitations in Italy, except for the Neapolitan church of Spirito Santo (Holy Spirit): a XVIII century work by M. Gioffredo.

Conclusion

Introducing many sometimes hazardous novelties in the technique of construction, not without difficulties and danger for the stability and with serious risk for their own safety, prestige, and for legal and economical consequences, on personal or institutional grounds, the three masters of roman baroque bravely experimented different original types of domes: from the more traditional, like S. Maria Assunta in Ariccia by Bernini, to the absolutely rivolutionary one, like Sant'Ivo alla Sapienza, by Borromini, to the brilliant novelty of Ss. Ambrogio e Carlo al Corso by Cortona.

The innovative aesthetical, technical and structural results obtained in such a singular and bold work of art, built by Cortona in his declining years, merge in an original impressive and stimulating architectural and urban image. The meaning of Cortona's efforts was to conceive the domes as a whole sculptured object of art, both as inside and outside, in which lightening has a central role, enhanced by the impressive reduction of the structure to a skeleton. The model elaborated by Cortona will become a paradigm for many generations of architects engaged in the construction of baroque and rococo domes throughout Europa.

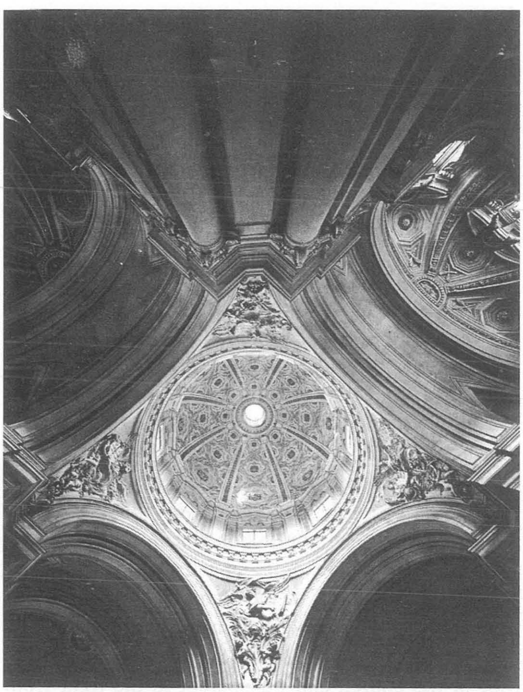

Figure 15
Rome, Ss. Luca e Martina: dome

Note

The theory; The intervention of Cortona on previous built domes, Cortona's new domes (Ss. Luca e Martina) by A. Cerutti Fusco. Cortona's new domes (Ss.Ambrogio e Carlo al Corso) by M. Villani. The remaining parts are by both authors.

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Robert Stephenson and planning the construction of the London and Birmingham Railway

Michael M. Chrimes

> It was the first of our great metropolitan railroads, and its works are memorable examples of engineering capacity. They became a guide to succeeding engineers; as also did the plans and drawings . . . When Brunel entered upon the construction of the Great Western line he borrowed Robert Stephenson's plans, and used them as the best possible system of draughting. From that time they became recognised models for railway practice. To have originated such plans and forms, thereby settling an important division of engineering literature, would have made a position for an ordinary man. In the list of Robert Stephenson's achievements such a service appears so insignificant as scarcely to be worthy of note.
>
> (Jeaffreson 1864, 1: 213).

Jeaffreson's modest final accolade highlights the significance of the London and Birmingham Railway. Robert Stephenson's appointment as its Engineer of the on 19 September 1833 (Directors 1833) marked a new stage in the general development of civil engineering in the British Isles. This paper will consider Stephenson's appointment in the context of civil engineering at that time, and the experience available within the profession. It will focus on how he organised the construction of the railway, and its impact on civil engineering generally.

In the early 1830s civil engineering was a profession moving to maturity (BDCE; Chrimes 2003a; Skempton 1996a; Watson). The term had been coined by John Smeaton about seventy-five years earlier and, following its foundation in 1818, the Institution of Civil Engineers had secured a Royal Charter in 1828. There had been skills shortages in civil engineering in years of high demand, and it remained difficult to obtain adequate training in the profession. It was not until the 1820s that the majority of practioners had received training explicitly as civil engineers, and not until 1841 that the Institution of Civil Engineers were to insist on this of its Members. By the standards of the time Robert Stephenson with his training and university education, was well prepared.

At the time of the construction of the Liverpool and Manchester Railway in the late 1820s there were many people around with more experience of civil engineering than the Stephensons. By 1833 the success of locomotive traction had changed the situation dramatically, and put the services of the Stephenson school of engineers in high demand. Robert Stephenson was free of other commitments and the Railway's Directors may also have felt Robert's youth might make it easier to tailor him to their needs.

The Route

Early proposals for a railway between London and Birmingham were projected by William James (1820) and (Sir) John Rennie (1825–26) and followed a more westerly route than that proposed for a rival company by Francis Giles in the late 1820s (Chrimes 2003b). It

was Giles' route which formed the basis of that developed by Robert Stephenson (1831–1833). Once the route had been identified surveying teams made surveys indicating the property ownership along the line in preparation for a Parliamentary bill. This failed to be enacted in 1832, largely due to opposition from property interests but was successful the following year. At that time most civil engineering works in Britain were privately financed, and required a parliamentary act before they could proceed. The London and Birmingham's Act was passed in May 1833.

Project Organisation

The Act represented a license to proceed —with the detailed surveying as a prelude to the purchase of land with raising capital, constructing the works, and operating the railway itself.

Over the previous seventy-five years three generations of engineers had met similar challenges and developed established procedures for carrying out civil engineering works, but rarely on the same scale. With major linear works of the London–Birmingham type the greatest obstacle to completion had often proved not the engineering challenges of the route, but rather raising the capital necessary. This had stalled works on Smeaton's Forth–Clyde Canal, Brindley's Oxford Canal, Rennie's Kennet and Avon Canal, and Jessop's Grand Junction Canal, as well as a whole host of lesser works (see table 1). More recently the Liverpool and Manchester Railway had sought an Exchequer loan for its works.

The timing of the London-Birmingham Act was fortunate in that the operating success of the Liverpool-Manchester Railway emboldened investors, the passage of the Reform Act promised political stability, and the economic cycle was on an upturn. As work proceeded circumstances began to change, wages and thus costs rose, and in the late 1830s there was a mini-economic crisis, which affected Brunel's work on the GWR; by then the London-Birmingham Railway had opened. One challenge, therefore, was to construct the railway as quickly as possible to enable investors to see a return on their capital before they lost heart.

As originally presented to Parliament , the line was 111 miles in length from Camden Town, London, to Curzon Street, Birmingham, with gradients nowhere exceeding 16ft per mile (1:330), and involved 12 million cubic yards of excavation and nearly 11 million cubic yards of embankments, as well as 6 viaducts, some 300 bridges and three long tunnels. It was on a scale rarely matched before or since. For comparison one can refer to table 1 for canal works, and table 2 below for other major civil engineering projects.

Table 1. Some Major Canal Works - 1760–1830

Project	Engineer	Years	(Engineering) Costs
Oxford Canal I	Brindley and Simcock	1769-1778	£200,000
Oxford Canal II	Barnes	1786-1798	£ 56,000
Oxford Canal III	Vignoles	1828-1834	£170,000
Forth and Clyde Canal I	Smeaton and Mackell	1768-1777	£164,000
Forth and Clyde Canal II	Whitworth	1785-1791	£140,000
Kennet and Avon Canal	Rennie	1794-1810	£860,000
Grand Junction Canal I	Jessop	1793-1805	£ 1.5m
Grand Junction Canal II	Barnes	1797-1805	See above
Grand Union Canal? III	H Provis and Bevan	1810-1814	£290,000
Caledonian Canal	Jessop and Telford	1803-1823	£855,000

Table 2. Major British Civil Engineering Projects - 1600–1830

Project	Engineer	Years	Costs	Approx. Value
Great Bedford Level	Vermuyden	1650-1656	£250,000	170m
Westminster Bridge	Labelye	1738-1750	£198,000	100m
Trent and Mersey Canal	Brindley and Henshall	1766-1777	£300,000	150m
West India Docks	William Jessop	1800-1806	£515,000	160m
Bristol Harbour	William Jessop	1804-1810	£470,000	140m
Plymouth Breakwater	The Rennies	1812-1850	£ 1.5m	400m
Sheerness Dockyard	The Rennies	1813-1830	£ 1.6m	400m
London Bridge	The Rennies	1824-1831	£425,000	105m
Liverpool and Manchester Railway	George Stephenson	1826-1830	£600,000	150m
London and Birmingham Railway		1833	£ 2.5m (original estimate)	450m

To guide him Stephenson had his own experience and observations on projects in which his father had been involved. He could also build on the precedents set by previous generations of engineers. In this the work of Smeaton was particularly significant. In the early 1830s there was little available in the way of engineering textbooks to draw upon (Skempton 1987) and Smeaton's published reports provided practical illustrations of engineering (Smeaton 1814). Stephenson later acknowledged Smeaton's influence: *«Smeaton is the greatest philosopher in our profession this country has yet produced»* (Smiles 1861: 2, 86)

Smeaton's Forth-Clyde Canal provided a model management structure for linear works. The project organisation of the London–Birmingham Railway mirrored this model. The route was divided into divisions under assistant engineers, with sub-assistant engineers and overseers responsible for the day-to-day supervision of shorter sections (Table 3). Each division involved a number of contracts, based on what was considered reasonable capital resources for a contractor. Generally a balance of cuttings and embankments was sought in each contract to minimise the need to haul over long distances. Separate contracts were drawn up for some major works and later works such as station buildings. Before contracts could be issued estimates were prepared to assess tenders properly, specifications and drawings prepared for inspection by contractors to enable them to price their work, and detailed land surveys carried out to enable land purchases to proceed. All this required staff, an opportunity for Stephenson to bring in experienced and trusted individuals who would share his workload.

Appointment of Staff

Stephenson's experience provided him with the opportunity to judge in general terms the qualities of staff he would require. Although many of George Stephenson's associates were tied up elsewhere, Robert was largely able to rely on people already known to him and experienced in railway work for senior appointments. The week following his own appointment, on 26 September, Stephenson made his first recommendations for engineering appointments: John Dixon and William Crosley as assistant engineers, and S. Bennett, J. C. Birkenshaw, E. Dixon and C. Fox as draughtsmen to work at the London end (Directors 1833). The next day he recommended T. L. Gooch as Assistant engineer, with John Brunton junior, at the Birmingham end (Birmingham 1833).

Stephenson was unsuccessful in his recommendation concerning John Dixon, who remained with the

Liverpool and Manchester Railway. . Further negotiations regarding salaries, and the appointment of George Watson Buck and Frank Forster as additional Assistant Engineers, followed in the next 3 months. This team of Assistant Engineers were to be in charge of construction until April 1837 when Thomas Gooch went to take charge of the construction of the Manchester and Leeds Railway . The management structure for construction can be seen in table 3. The engineering staff on the line eventually, in late 1837, numbered 55 (CEAJ 1837), and details of known names and appointments can be seen in table 4.

Of those not known to him personally Buck and Crosley had considerable experience of construction, and Buck also had a reputation for his structural use of iron (BDCE 2002). Fox's engineering experience was of a more mechanical nature, and he clearly had a commanding presence (Conder 1983, 11–12). Generally Stephenson's management technique was to appoint young aspirant engineers to junior positions, entrusting them with more responsibility and independence as they proved themselves. He later wrote to Brunel, speaking specifically of G. H. Phipps:

> I have always met that by reposing the utmost confidence in him taking care of course that my principles of conducting operations were adhered to . . . (Stephenson 1838)

Table 3.

Station	Miles	Principal Works	Engineering Staff Sub-Assistant*	Assistant
Euston	0			
Camden Town Depot	1	Retaining walls		Charles Fox
		Primrose Hill Tunnel	F. Young	
Watford	18	Watford Embankment and Colne Viaduct	T. Jenkins	George W. Buck
		Watford Tunnel	Captain Cleather	
Tring	32	Tring Cutting	S. S. Bennett	
[Denbigh Hall]	48		E. Jackson	William Crosley
Wolverton	52	Wolverton Embankment and Viaduct	T. Gandall	
	58			
Blisworth	63	Blisworth Cutting	G. H. Phipps	Frank Forster and G. H. Phipps
Weedon	70			
	79	Kilsby Tunnel	C. Lean	
Rugby	83		H. Lee	
		Avon Viaduct	J. Brunton	Thomas Gooch and Frank Forster
Coventry	94			
		Beechwood Tunnel	B. L. Dickinson	
Birmingham	112	Rea Viaduct		(after April 1837)

Table 4. London and Birmingham Railway - Engineering Staff

Name	Date of Appointment	Annual Salary	Role
George Aitchison	January 1834 January 1837	£150 £270	Clerk Architect intermediate stations
Bagster	1836		Superintendent of station layouts
William Baker			Pupil of G. W. Buck
J. Bennett?			
S. S. Bennett	1833 1835	£200	Draughtsman Sub-Assistant Engineer, Tring
J. J. Berkley	(1837)		Pupil of G. P. Bidder
George Parker Bidder	17 September 1834		Assistant in drawing office
John Cass Birkenshaw	November 1833 May 1834 19 November 1834	£200 £400	Draughtsman, staking out London area Drawings, etc., Birmingham Division Manager, Primrose Hill direct labour
P. Browne	?1837?		Assistant?, Coventry
John Brunton	 11 October 1833	 £250	Assists in preliminary surveys Sub-Assistant Engineer, Birmingham Division, Avon Viaduct
William Brunton	1830-1831		Surveyor/Resident, London end
George Watson Buck	December 1834	£600	Assistant Engineer, 'B' (Watford Division)
Budden	May 1834 1836	£100	Office Assistant
D. Carter	1836		Draughtsman, Coventry, Clerk of Works, Euston Station
Charles Frederick Cheffins			Assisted with drawings
Captain Cleather	October 1834	£200	Sub-Assistant Engineer, Nash Mills, Tring
Francis R. Conder	1834		Pupil of Charles Fox
William Crosley	1833		Assistant Engineer, 'C' Division
William Crosley (junior)	1833 (6 April 1836)		Pupil (Assistant) of father, in drawing office
Bernard L Dickinson			Sub-Assistant Engineer, Birmingham September 1835 1837
Edward Dixon	1832 September 1833 7 February 1834	 £200	Assisted in surveys Draughtsman Sub-Assistant Engineer
R. Dixon			Draughtsman
Robert Benson Dockray	December 1835 1837 7 March 1838	 £300	Assistant Sub-Assistant Engineer Resident Engineer, Birmingham Division
Mark Faviell (junior)	?1835		Sub-Assistant Engineer
Frank Forster	1 November 1833 December 1833 April 1837	£500 £600	Assistant Engineer, Weedon Assistant Engineer, Birmingham
George Foster	5 May 1834 1837		Draughtsman, St John's Wood Assistant, Coventry office
Fowler	1837		Assistant, Coventry office
Charles Fox	1833 1834 1835 August 1837	£200	Draughtsman Sub-Assistant Engineer, Watford Assistant Engineer, Euston Extension Resident Engineer, London–Wolverton
John Gandell	2 September 1835		Sub-Assistant Engineer, Wolverton

Table 4. (Continuation)

Name	Date of Appointment	Annual Salary	Role
Thomas Longridge Gooch	October-November 1831 1832 11 October 1833 December 1833	 £500 £600	Surveys of route Estimates and further surveys Assistant Engineer, Birmingham District (Coventry)
Conrad Hanson	4 October 1837	£150	Office Assistant
William Hanson	1836?	£ 50	Office Assistant October 1837
George Harris	[1837]		Pupil of Charles Fox
Thomas Elliot Harrison	1830-1831		Assistant on surveys
David Hodgson	August 1837		Assistant, Wolverton
Edward Jackson			Sub-Assistant Engineer
Timothy Jenkins			Sub-Assistant Engineer
King			Assistant to Aitchison
Charles Leam	1 August 1834 1835	 (£400 from June 1836)	Draughtsman Sub-Assistant Engineer, Kilsby Tunnel
Peter Lecount	1832 May 1834	 £150	Traffic forecasts Clerk, Engineer's Department Sub-Assistant Engineer, Birmingham Division
Hedworth Lee	1835 1837		Draughtsman Sub-Assistant, Weedon
William Price Marshall	1835		Draughtsman
Sturges Meek	1833 December 1836		Pupil of George Stephenson Sub-Assistant Engineer
George Mackay Miller	1833?		Draughtsman
M. Monteleagre			Sub-Assistant Engineer
John Nash	1834	c.£150	Overseer
Paul Padley	September 1833		Stakes out line
Perry	[c.1837]		Assistant, Berkhamstead
George Henry Phipps	1833 1835	£200	Draughtsman Sub-Assistant Engineer, Weedon
Robert Rawlinson	December 1834?		Assistant at Blisworth, 1836
John Reid			Sub-Assistant Engineer
Luke Richardson	1836		Overseer
J. Riches	[1837]		Assistant, Coventry office
James Routh	[1830s]		Pupil/Assistant to G. P. Bidder
William Routh			
William Rudge			Assistant, Coventry office
J. Sharpe			Clerk of Works, Camden Station
Thomas Macdougall Smith	1830s		Assistant
Herbert Spencer	10 November 1837	£80	Pupil of Charles Fox
Robert Stephenson	1830-1833 September 1833	 £1,500	Joint Engineer, Parliamentary planning Engineer-in-Chief
Stokes			Draughtsman, St John's Wood
Francis Thompson	December 1838		Assistant architect
Richard Townsend	1837		Sub-Assistant Engineer, Tring
Francis Mortimer Young	1833 10 December 1834	 £200	Draughtsman, Coventry office Sub-Assistant Engineer, Primrose Hill Tunnel

The team's first task was to stake out the actual line of the railway. The Act allowed deviation within a band 100 yards wide and gave railway staff authority to enter property to fix the route. Stephenson believed that by employing experienced engineers he could save on land surveying costs (London, 26 9 1833); it would also give his senior staff first hand acquaintance with the route. He himself had walked the line 12 times by May 1834 (Conder 1983, 14).

Levels were taken every chain (22 yards) along the line, enabling the preparation of sections, and thus quantities. Trial shafts were sunk at several locations to obtain additional information on the strata to that gathered prior to the Act, and providesamples for the contractors. These investigations led to a modification of the design slopes, and a consequent increase in costs.

As the route was fixed by the engineers in the field, contract drawings could be made and specifications drawn up. Common sense suggested that contracts should be let at the London and Birmingham ends first as there traffic was likely to be greatest and could generate income to offset against expenditure (Rastrick 1833). Thereafter, the most difficult contracts namely those at Tring and Kilsby Tunnel were prepared so delays there would not hold up the opening of the whole line. Table 6 indicates the details of the contracts, and when they were awarded.

The process was apparently straightforward, but from contemporary sources it is clear Stephenson was short-staffed in early 1834. His staff included, aside from himself, 4 assistant engineers, several of whom had pupils (table 4), and 4 sub-assistants, and 8 known office staff. It took two man-days to prepare a drawing and 2,000 drawings had to be prepared for the line, the equivalent of more than 10 man years work (Lecount 1839). Stephenson himself spent at least three days on the Rea viaduct for which three drawings were prepared, He personally supervised the drawings, and was also responsible for the specifications (Stephenson 1834). Although Brunton speaks of two shifts of 20 drawing office staff, this must surely refer to a later period. He claimed to have worked twenty hours (i.e., 2 man-day) shifts with only one night's sleep for a fortnight, personally delivering the Birmingham drawings for the inspection by the directors on 4–5 July 1834 (Brunton 1930, 36). One suspects others were as busy. Pupils were probably all drafted in to meet deadlines and additional draughtsmen taken on at short notice.

This meant that there was an initial «crisis» on the critical path; the human resource dictated how quickly contracts could be let and work commenced. The Birmingham directors expressed their disquiet at the delay in starting work at their end of the line (Birmingham, 16 5 1834) Pressure became more acute as work began on site requiring supervision, and preparations for an extension act to Euston from Camden began in the autumn of 1834.

Selecting the Contractors

Once the specifications and contract drawings had been prepared the contracts could be put out to tender. A senior engineer consulted by the Company, John Urpeth Rastrick, recommended putting the contract out to tender in large lots, since «extensive contracts . . . become worth the attention of men of capital who should they unfortunately find that they have taken the work for too small an amount, or should the seasons become unfavourable for the execution thereof, so that they may run the risk of losing money by the contract they will still go on and complete the work sooner than suffer the least imputation on their character or respectability . . . whereas when work is let in little petty contracts they are generally taken by men of no capital whose security is good for nothing, and as soon as ever they discover or think that they have made an imprudent contract, begin immediately to have recourse to every expedient to get rid of it . . . » Moreover «I have always found that the work was much better done and that everything went on with more expedition and the Engineer's orders all were punctually attended to when the contractor had a fair and liberal compensation from his contract» (Rastrick 1833, 30)

Henry Robinson Palmer, founder of the ICE, was of a similar mind (Palmer 1833). Engineers discussed this approach in their evidence to the Select Committee on the Southampton Railway (Southampton 1833). While Rastrick's theory may have been sound there were practical problems in finding contractors with the financial resources to take on such lots. A number of contractors with many years of experience in civil engineering, but only a

handful had taken on contracts of more than £100,000 in value before this time, which may have cautioned a circumspect approach. Perhaps there was a feeling that the domination of public works contracting by McIntosh and Banks in the 1820s may have worked against the client's interests (BDCE). In practice contracts were let in batches to suit project control, the office workload dictated the timetable, with lot size determined by convenience and the engineering challenges anticipated. Most were for 4–6 mile lengths, excluding special structures. For the Tring contract major contractors were invited to tender,[79] anticipating the heavy work involved, but elsewhere there was a free-for-all.

Stephenson and the Board were aware of the risks attached to accepting the lowest tenders from inexperience or under capitalised contractors, and contractors were expected to provide 10% sureties, and operate with a retention (Townsend). Tables 5 & 6 provide details of the contracts, date, contractors, estimates, tenders and out-turn. Table 7 gives an indication of the contractors previous experience. The line was broken down into smaller lots than Rastrick and Palmer had advised and Stephenson seems to have determined on a value of 30–50,000 pounds (London, 12.11.1834). However as a few contractors took several lots, Rastrick's suggestion could have been met in practice, and arguably Stephenson did not take full advantage of the contracting experience available.

The most striking things about the tables are the number of contractors who «failed». Eight contracts of 30 were completed by the Company, and in 4 other cases contracts were either re-let or completed by another contractor on the line. Interestingly few of these enterprises continued contracting into the 1840s. Perhaps some contractors were too old to meet the physical challenges involved, while their younger assistants, often their children, lacked the necessary experience.

Expenditure and Cost Control

The eventual cost of the London and Birmingham Railway on opening at c.£5.5 million was twice that of the Parliamentary estimates. In some quarters Stephenson was heavily criticised for extravagance and the inaccuracy of his estimates, but generally the view of his contemporaries was one of respect for his achievement, a view no doubt shaped by the financial success of the railway.

Table 8 displays how estimates changed from 1831 as construction proceeded, revised estimates made in the light of escalating expenditure, and final costs when the line was opened.

The costs of John Rennie's proposals were never published, but estimated for civil engineering work at £1.25 million. The detailed estimates for parliament were prepared by Stephenson and Gooch, presumably based on the Liverpool and Manchester experience. In almost every category estimates were grossly exceeded, and in this context those within Stephenson's direct control, particularly the civil engineering work, are not disproportionately costly (Table 8). Bearing in mind that the tenders came in within Stephenson's estimates (table 6), and most other leading engineers broadly concurred with Stephenson's figures, contemporary explanations for the cost overruns must be taken seriously.

As detailed at the end of 1836 these were the engineering problems posed by the difficult ground conditions on the Primrose Hill, Kilsby and Blisworth contracts, the increase in the volume of earthworks caused by modifications to the design slopes, all within Stephenson's sphere, unexpectedly high land prices, the additional costs of stations, and rising iron prices. The price of land and iron (Table 8 and fig 1) while attributable in part to the demand from the railway, were beyond its control. The increased costs of the stations were attributed to the improving prospects for railway traffic, in part due to the establishment of further connecting lines, and the success of the railway on opening; this also led to the purchase of more locomotives and rolling stock. It is of some interest that in some earlier estimates Stephenson had perhaps made more realistic estimates regarding the stations.

Expenditure is displayed graphically in figure 2 With regard to progress on works, the initial idea was to let the London and Birmingham section contracts first, to bring in income, and then those anticipated taking longest. Each contract specified quantities of earth to be shifted by specific target dates. Almost immediately problems with this approach were exposed due to the frailty of the contractors; this put

Table 5. Contracts

Contract	Length in Miles	Contractor	Date	Price (£)
Euston Extension	1	W. and L. Cubitt	Dec 1835	76,860
1B Primrose Hill	$5\,{}^{3}/_{4}$	T. Jackson*	May 1834	119,987
2B Harrow	$9\,{}^{1}/_{2}$	Nowell and Sons	May 1834	110,227
3B Watford	5	James Copeland	May 1834	117,000
4B King's Langley	$2\,{}^{1}/_{4}$	W. and L. Cubitt	September 1835	38,900
5B Berkhamsted	$4\,{}^{1}/_{2}$	W. and L. Cubitt	September 1835	54,660
6B Aldbury	$2\,{}^{1}/_{2}$	Richard Parr	September 1835	14,500
1C Tring	6	Thomas Townshend*	September 1834	104,496
5C Leighton Buzzard	3	James Nowell	September 1835	38,000
6C Stoke Hammond	4	E. W. Morris	September 1835	39,303
7C Bletchley	$3\,{}^{3}/_{8}$	John Burge	September 1835	54,500
2C Wolverton	5	William Soars*	October 1834	67,732
4C Wolverton Viaduct	${}^{1}/_{8}$	James Nowell	February 1835	25,226
3C Castlethorpe	$4\,{}^{1}/_{2}$	Craven and Sons	October 1834	49,735
1F Blisworth	5	William Hughes*	February 1835	112,950
2F Bugbrooke	5	John Chapman	February 1835	53,400
3F Stowe Hill	$1\,{}^{1}/_{4}$	John Chapman	February 1835	23,050
4F Weedon	$1\,{}^{1}/_{8}$	Edward Beddington	May 1835	23,090
5F Brockhall	$3\,{}^{1}/_{8}$	J. and G. Thornton	May 1835	34,157
6F Long Buckby	$3\,{}^{5}/_{8}$	J. and G. Thornton	May 1835	42,582
7F Kilsby Tunnel	$1\,{}^{3}/_{8}$	Nowell and Sons*	May 1835	98,988
7G Rugby	$5\,{}^{1}/_{8}$	Samuel Hemming*	February 1835	59,283
6G Long Lawford	$3\,{}^{1}/_{4}$	W. and J. Simmonds	February 1835	20,330
5G Brandon	$4\,{}^{1}/_{4}$	Samuel Hemming*	February 1835	40,000
5G Avon Viaduct	$1\,{}^{1}/_{6}$	Samuel Hemming	November 1835	7,970
4G Coventry	$7\,{}^{3}/_{4}$	H. Greenshields*	November 1834	101,700
3G Berkswell	$4\,{}^{1}/_{2}$	Daniel Pritchard	November 1834	53,248
2G Yardley	$7\,{}^{1}/_{2}$	Joseph Thornton	August 1834	68,032
1G Saltiey	$1\,{}^{7}/_{8}$	James Diggle	August 1834	32,878
1G Rea Viaduct	${}^{1}/_{8}$	James Nowell	August 1834	13,644
* work later taken over by Company				1,698,681

Table 6. Contract estimates and final expenditure

Contract	Parliamentary est 1832	Engineers' est 1834	Contract price	Revised contract price	Dec 37 expend	Final expend	% overrun on estimates
1b	93998	120668	119987			280014	232
2b	88436	104089	110277			144574	139
3b	105738	102944	117000			138219	134
4b	33775	41114	38900			57386	139.5
5b	41562	58648	54660			65002	111
6b	15795	16694	14500	16694		25134	150.5
1c	86006	*98298*	**104496**			144657	147
5c	32422	**33502**	**38000**			43162	129
6c	42048	**43869**	**39303**			42345	96.5
7c	40468	**48398**	**54500**			61071	126.
2c	73920	*75081*	**67732**			107765	143.5
4c	31150	*27163*	*28132*	25226		28964	106.6
3c	43271	*45224*	*48414*	49735		71873	159
1f	113400	*110097*	**112950**			184301	167
2f	45455	*56414*	**53400**			65013	115
3f	19887	*24596*	**23050**		25571	31536	128.
4f	80104	*28217*	**23090**	26150	25860	31442	111
5f		*40000*	**34157**			50583	126
6f	39297	*46293*	**42582**			48256	104
7f	84815	*102174*	**98988**			291030	285
7g	55971	*72684*	**59283**			93384	128.
6g	23119	*26882*	**20330**		22740	25893	96
5g	38775	*43648*	**40000**		42272	55090	126
avon	6519	*8031*	**7979**		8421	8621	107
4g	103335	*108898*	**101700**		127488	150496	138
3g	63264	*50252*	**53248**		56281	62738	125
2g	70512	*66842*	**68032**		68127	78131	117
1g	32883	*35057*	**32878**		34862	38707	110
rea	9489	*13380*	**13644**		14928	15505	116
Euston	83810	*83810*	**76860**			91528	109
Total	1599224	1732967	1698072	1698681		2532420	146

Table 7. Contractors Experience, etc.

Name	Years Experience	Principal Works
Edward Beddington	no information	
John Burge	c.5 years	St. Katharine's Docks
John Chapman	no information	
James Copeland	c.10 years	Liverpool and Manchester Railway, Leicester and Swannington Railway
Hiram Craven	25 years	Union Canal, Hull Junction Dock
W. and L. Cubitt	15 years	Fishmongers Hall
James Diggle	?5 years	?Warrington and Newton Railway
Hugh Greenshields	c.10 years	Sankey Viaduct, Liverpool and Manchester Railway
Thomas Harding	?only as labourer	Leicester and Swannington Railway
Samuel Hemming	15 years	Bombay Engineers 1819
William Hughes	30 years	Caledonian Canal
Thomas Jackson	10 years	Assistant to Grundy, London Building Contractor
William Mackenzie	25 years	Birmingham Canal
E. W. Morris	15 years	Holyhead Road, B&LJ Canal
James Nowell	20 years, mason	Various churches
Joseph Nowell	20 years, mason	Macclesfield Canal, St. Helens Railway
R. Parr	mason	Newcastle and Carlisle Railway
Daniel Pritchard	20 years	Harecastle Tunnel, Trent and Mersey Canal
W. and J. Simmonds	no information	
William Soars	c.15 years	Macclesfield Canal
George and James Thornton	10 years	Liverpool and Manchester Railway
Joseph Thornton	5 years	Liverpool and Manchester Railway
Thomas Townsend	40 years	Birmingham Canal

the construction timetable under pressure. The timetable was also upset by delays in preparing the drawings and specifications. The early prospects of revenue were diminished.

As early as September 1835 Stephenson was under pressure to improve progress, and he made suggestions for speeding up work at Primrose Hill (London, 22 9 1835) At the end of December he provide estimates for expenditure in the following year (London, 29 12 1835) Following his report on progress on the Southern section at the end of January a special committee was set up to inspect the works, and report on the state of progress (London, 27 1 1836). There was concern about running out of finance, and contract targets not being hit, with the first contracts due for completion that June. It was clear that under existing conditions there was no prospect of opening the line on time A further inspection was crushing in its condemnation of the contractors Copeland and Harding:

Table 8.

	Feb 1831 Est	Parl Ests 1833	Contract tenders 1835	1836 ests	June 1840	1840 as % of 1833
works	1671574	1558852	1698681	2528890	3927647	252
land	423100	250000	250000	506500	706152	169
parliament	50000	73000	**72869**	84869	**72869**	99.8
permanent way	547968	315900	315900	693822		0
rolling stock	168000	61000	61000	253715	336097	196
other costs		99191	99191	224277	295609	298
contingencies	150000	294648	294648			0
buildings	71000	19600	19600	154421	360000	507
Total	3081642	2672191	2811889	4446594	5698374	213

> . . . the system now acted upon by the contractors is unskilled and altogether inefficient for the proposed object . . . Mr Copeland has been absent for the railway above 6 weeks and does not appear to have the power to act when present. Mr Harding is little better than a labourer and has no authority.(London 1836, 205–209, 212–216)

Stephenson came under increasing pressure to perform. By the end of 1836 the Board realised that the financial estimates would have to be revised, in the light of slow progress, and also escalating costs. The crisis of confidence in Stephenson's ability had reached its peak over the construction difficulties caused by quicksand in the Kilsby tunnel. As Stephenson determined his course of action a Company Secretary Captain Moorsom arrived, and was fortunately impressed enough to persuade his colleagues it was not necessary to call on another engineer (Jeaffreson, 203–204). This said, directors' visits continued and in April 1837 it was decided to assign individual directors to monitor each unfinished contract (London, 12 4 1837). This could be said to be undermining the engineer's authority, although in the end it may have enhanced it as his engineering judgement, exposed to the utmost scrutiny, was vindicated.

One can see the fates conspiring against Stephenson. Having promised a, delayed, opening at the end of 1836, the weather conditions caused him to withdraw all forecasts in December (Engineers, 12 1836). In the end the effort involved was enormous, with regular night work endorsed by the directors in the pursuit of an opening of the railway. The additional costs were met by additional borrowing and calls on shares permitted by supplementary acts of Parliament.

From extant records it is known that George Aitchison was responsible for monitoring project costs in the London office, Stephenson produced a regular account of work done and to be done, and latterly printed reports were provided for the Birmingham Committe. Project control could not have been tighter.

The civil engineering industry has an unenviable public reputation for cost overruns on major projects. The Channel Tunnel construction costs escalated

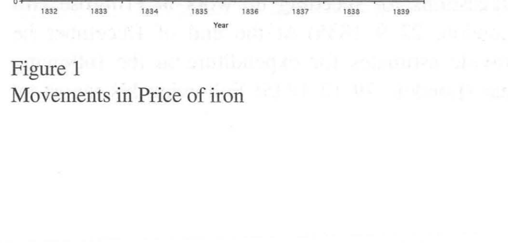

Figure 1
Movements in Price of iron

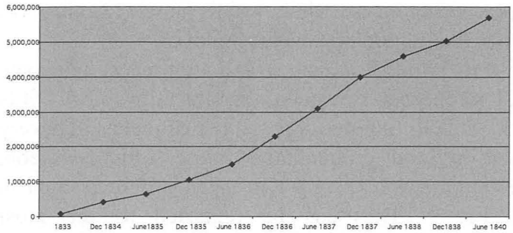

Figure 2
Expedinture en London and Birmingham Railway

from an estimated £3.8 billion to £5.8 billion in real terms; in contrast the civil engineering work on the Millennium Dome was delivered ahead of time and below budget. Cost overruns more typically occur on prototype projects such as the Thames Tunnel (estimates £200,000; final costs c.454000), particularly where ground problems are experienced. The scale of the London and Birmingham, particularly the Kilsby Tunnel, exposed its shareholders and engineers to such a risk. Despite the thoroughness of Stephenson's approach he can be criticised. Although not evident from most other engineers' parliamentary evidence the costs of gentler slopes could have been anticipated. John Rennie snr's experience at the Mint, and problems at Highgate archway had revealed difficulties in working with London Clay. Another alignment could have been selected away from Kilsby, as demonstrated by Sir John Rennie's route, which was very similar to parts of the later GWR route to Birmingham, though longer than Stephenson's. Stephenson and Gooch had actually surveyed an alternative. However generally Stephenson's work and project control was exemplary and it was the complexity of the project that determined the resources required.

Relationship with contractors

Stephenson generally did well both for the contractors and the Railway Company. He was prepared to negotiate on prices, but less compromising on lack of progress. As work progressed Stephenson increasingly recommended the Company took over contracts. This was anticipated in a letter to Moorsom, on 30 November 1835, which encapsulates his attitude to the contractors-:

> «with contractors who understand the work and with adequate capital you will not have to do any more than urge them on and serve notices. With others the Company may need to enter the works and provide materials, as you have already had to in two cases». (Engineers, 30 11 1835).

One can have sympathy with him; at much the same time Forster was reporting:

> «There seems a sort of fatality among our contractors. Nowell has been seriously ill and is still weak. Chapman is very ill of inflammation in the region of the heart, and poor Hughes is lying in an almost helpless state at Northampton, of a paralysis of the limbs». (Engineers, 30 12 1835)

Having persevered with William Soars in the face of all kinds of financial and engineering difficulties he finally requested the London Committee «would authorise him to serve Mr Soars with notice of the termination of the contract, as the only resource which the Company had left . . . » (Directors, 17 5 1837)

Generally he was fair to the contractors and paid them in full for their work. Although they were only paid monthly, rather than fortnightly which was more common e.g on the GWR, he was aware of the financial problems this could cause. He recommended the Board advance money for them to obtain wagons and to purchase locomotives to make them available for the contractors to move material more effectively.[1] The whole system relied upon trust and vigilance. Once work had begun in earnest it was impossible for Stephenson to personally supervise all the works. He had to trust his sub-assistants, and concentrate his efforts where there were problems, most notably at Kilsby. This approach seems to have worked well.

Site Investigation

The quantities of earthwork made it necessary to use exceptional care in deciding on the slopes to be adopted in cuttings. A thorough site investigation was therefore carried out as a preliminary to parliament

with about 45 borings, mostly from 10 to 20m deep; well records were examined and observations made in quarries and road or canal cuttings close to the line. Further investigations followed the passage of the Act, leading to a modification of design slopes, reducing their steepness, and causing a consequent increase in costs.

Design of the works

With regard to the design of the bridges and station structures Stephenson himself was heavily involved in the detail of the drawings and specification. He discussed his ideas with his assistants, thus on finds him discussing the details of skew bridges with Buck in February 1834 . . . (Stephenson 1834). His discussions were not confined to senior colleagues as he readily accepted Charles Fox's contributions to the design of the iron roof structures and bowstring bridges. Moreover, despite the attention paid to the specifications, one finds him prepared to modify the designs, and recommending increasing the costs where necessary, as with the early decision to substitute roman cement for lime mortar in the bridges (Engineers, 28 5 1835), influenced no doubt by problems at Primrose Hill tunnel.

The station buildings, excluding the platforms and train shed roofs were, as normal for railways of the time, designed by the architects, Hardwick, Aitchison and latterly Francis Thompson. The detail of the station layouts were organised by Bagster. Stephenson concentrated on the operating side, providing general plans and instructions on what he expected, and offering general guidance, insistent that the stations could accommodate all anticipated traffic.

Construction

The first three contracts were let in May 1834; another eight had followed by November, and by February 1835 work was proceeding on more than half of the total length of the line. Work began in June 1834 and a year later work was in progress on twenty contracts with 4,000 men employed. All contracts had been let by November 1835 and the next month work started on the extension from Camden Town to Euston. During the following two years, with work proceeding on the whole length (now 112 miles), as many as 12,000 men were employed.

From specifications, working drawings and some of Stephenson's evidence on later railway Bills, it is possible to find the slopes at which practically every cutting and embankment were actually made. Examples are given for a dozen different strata in Table 9. This can be regarded as the most comprehensive, and most advanced, set of data for the period; several changes had been introduced since 1832:

Clearly Stephenson gave a good deal of thought to his clay slopes, prompted by Rastrick's advice and almost certainly influenced by Parnell's *Treatise on Roads* published in 1833,[2] as well no doubt by his own further enquiries. The effect of these changes was to increase the projected total volume of excavation to rather more than 12.5 million cubic yards, with perhaps about 11.5 million in the embankments with consequent increase in costs. While many modifications had been introduced before the contracts were let, one also comes across examples, as at 4G , when this was done shortly afterwards, perhaps in response to better knowledge of the ground. (Engineers, 4 March 1835). Practical problems in construction dictated further changes, as at Wolverton embankment. Ultimately perhaps 14 million cubic yards of material were excavated, an obvious source of the cost escalation.

Construction of embankments was generally carried out at full height, i.e. by tipping material at the end of an embankment until the final height was reached, rather than tipping at several levels simultaneously. The latter method had some attractions as it offered the possibility of working in more than one place simultaneously and less likelihood of subsidence, but in practice it proved difficult to manage such operations. A typical rate of progress on the Watford embankment was c. 190.000 cubic yards placed a year, and for Willesden embankment c. 160,000 cubic yards a year. More material could be removed if it was being excavated to spoil, and at Tring the average rate was 400,000 cubic yards of excavation a year.

By a concentrated effort probably without parallel hitherto in the history of civil engineering, and not surpassed for a long time afterwards, the line was opened from Euston to Tring (32 miles) in October 1837, from Birmingham to Rugby (29 miles) in April

1838, and throughout in September 1838. Overall about 3.1 million cubic yards of material were excavated a year, muck shifting on a scale not repeated elsewhere in the UK until the Great Central Railway some 60 years later (Skempton 1996b).

CONCLUSIONS

The successful completion of the London and Birmingham Railway in June 1838 consolidated the reputation of Robert Stephenson among his contemporaries as the leading civil engineer of his generation. It revealed the frailty of even the most experienced contractors, and a full range of difficulties posed by earthworks and tunnelling. While one can criticise a route selection which involved Kilsby Tunnel, close to an alignment which had already caused problems for canal builders, and the initial recommendations regarding earthworks, one can only admire, like most of his contemporaries, the scale of the achievement. Many of the cost increases stemmed from changes to the original brief to accommodate more traffic. It is arguable Stephenson should have recommended gentler slopes for the cuttings and embankments, and thus anticipated additional costs. Certainly past experience could have been used to justify this approach, but many engineers supported his recommendations. Many of these specifications had been altered before construction began, suggesting a flexible approach to engineering decisions.

The impact of the London and Birmingham Railway on the engineering literature of the time can be likened to that of Smeaton whose published record had guided the previous generation of engineers. Drawings and specifications formed major proportions of Brees' *Railway Practice*, and Simms *Public Works of Great Britain*, available as a model to all the engineers active in civil engineering at the time, «time-saver» standards for the railway age (Brees 1847; Simms 1838). Dempsey's papers on railway engineering for the

Table 9. Examples of cutting slopes on the London and Birmingham Railway

Stratum	Cutting	Date of Contract	Maximum Depth		Slope
			m	ft	
London Clay	Primrose Hill	May 1834	12	42	3:1
London Clay	Kensal Green	May 1834	9	30	2:1
Reading Beds Clay	Watford Heath	May 1834	12	41	2:1
Oxford Clay	Denbigh Hall	October 1834	13	45	2:1
Upper Lias Clay	Bugbrooke	February 1835	14	47	2:1
Lower Lias 'shale'	Church Lawford	February 1835	9	30	1 $^{1}/_{2}$:1
Keuper Marl	Yardley	August 1834	14	45	1 $^{1}/_{2}$:1
Lower Chalk	Tring	September 1834	17	57	1:1
Upper Chalk	Watford	May 1834	18	63	$^{3}/_{4}$:1
Lower Greensand	Linslade	September 1835	18	60	$^{3}/_{4}$:1
Keuper Sandstone	Berkswell	November 1834	17	55	$^{3}/_{4}$:1
Great Oolite	Blisworth	February 1835	17	52	$^{1}/_{4}$:1
Limestone					
Carboniferous	Beechwood	November 1834	17	54	$^{1}/_{4}$:1
Sandstone					

Royal Engineers were based heavily on London and Birmingham practice, subsequently published as a monograph through three further editions (Dempsey 1855). Practical experience of the construction of cuttings and embankments was reflected in papers and discussions at the Institution of Civil Engineers and elsewhere, while the illustrations of Bourne, and the texts of Roscoe and Lecount provided vivid images of the construction of the line (Bourne 1839; Lecount 1839; Roscoe 1839). Neither Locke nor Brunel served as such as exemplars to the profession.

Though Stephenson and his engineers experienced considerable troubles of a geotechnical nature, these must be seen in context of a project of unprecedented magnitude with regard both to the scale and number of works involved. Some of the problems could not possibly have been foreseen; all were satisfactorily solved, and for mile after mile many of the huge cuttings and embankments and viaducts never gave any trouble at all. Indeed Thomas Gooch regarded the northern section for which he was responsible as straightforward work (Gooch).

Notes

1. This had been done on the Liverpool and Manchester Railway. By 1833 contractors were purchasing their own locomotives on other contracts.
2 H Parnell. A Treatise on roads. London: Longman, 1833. Thomas Telford had spent several days in 1833 on Parnell's book which reflects his practice, particularly on the Holyhead Road. Slopes recommended (pp.80–87) for cuttings, except in stone, never less than 2:1 to admit sun and wind to the road. He recommended slopes for cuttings and embankments in London/plastic clay 3:1, chalk or chalk marl 1:1, solid sandstone 1:4, but if sandstone strata was mixed with marl the safe slope could vary between 1–5:1 and 1:4 according to the inclination of the strata; for Oxford clay 3:1–2:1, solid limestone 1:4, limestone and clay 1.5:1–2:1; granite, slate, etc., 1:4. Parnell also emphasised the need for good drainage, sod cover, and laying embankments concavely rather than convexly, i.e., from the outside in.

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The «Cuba» near Castiglione in Sicily. A self-supporting vault made of volcanic stone

Pietro Copani
Laura Buonanno

The Sicilian Medieval architecture is widely recognized as rich of different mixtures and influences, that affected features and spaces of the civilian buildings, and particularly the religious ones.

The wealthier period on this subject is certainly the one beginning with the conquest of the island by Normans after the Muslim rule (second half of XI century) and carry on for more than two centuries and half, until the end of Swabian dinasty of Frederick II in 1266.

The long Muslim presence in Sicily (from 827 to the Norman invasion led by Roger I) influenced the characters of architecture in the following centuries, by mixing its features with the surviving Byzantine traditions (expecially significant in the eastern part of the island) and with the European trends brought by the new sovereigns.

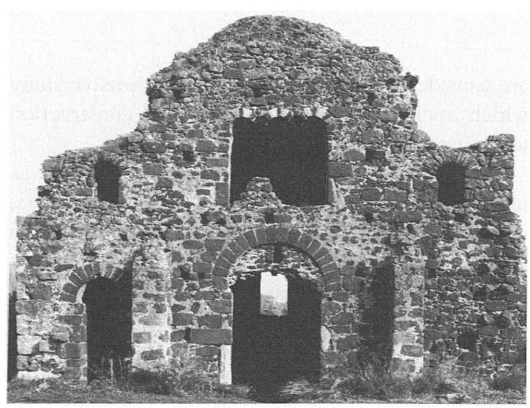

Figure 1
Santa Domenica church. Western façade in 1999. (Photograph courtesy of Arch. Rosario Musumeci)

The problem of dating Cuba

The building studied is a little church, laying ruined in the country-side at the north base of Mount Etna. It is called «Santa Domenica» (or «San Domenico») but this dedication seems to be more recent than the real foudation of the church: the first time this place-name is found, is on a map dated 1891, with the exact name of «Molini San Domenico» (Saint Dominick Mills).[1]

The «Santa Domenica» church is commonly called «Cuba»: probably this denomination has helped in the misdating of the building during the Byzantine period of the island (VI–IX centuries, and so the name should derive from the Islamic period after the building of the church).[2] Many scholars (Sardo Sardo 1910; Freshdfield 1918; Lojacono 1936; Bottari 1939; Pace 1949; Giglio 1997), linked the Cuba with a lot of little *cellae trichorae* in this part of Sicily. Some of these buildings are laying also few kilometres far from the Cuba (Lojacono 1960a, Giglio 1992a), and date back to the Byzantine period, but they are basically different from the building we

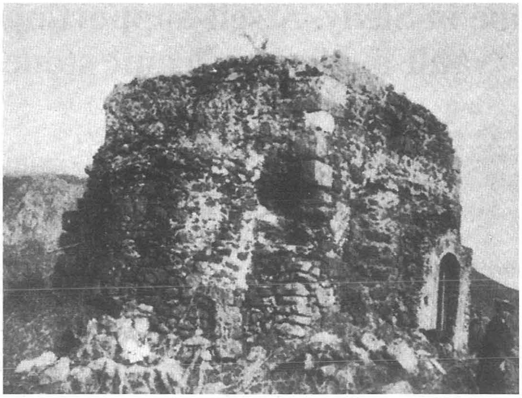

Figure 2
The Cuba near Malvagna, Sicily. It is dated back to the V–IX centuries. (Agnello 1952)

are considering, according to a comprehensive study which includes the analysis of the construction techniques.

The Cuba, abandoned several centuries ago, was used as a sheep-fold until 1959, when repairs, conducted by the Soprintendenza ai Monumenti della Sicilia Orientale, gave the building the present appearence (Lojacono 1960b, Lojacono 1963).

In the whole XX century only two times the Cuba was defined as a medieval building, far from the ones derived from the roman architecture. The first to write about a possible datation after the X century was Stefano Bottari, and this is a part of the description he made about the Cuba:

> La capziosa ingegnosità di questa costruzione, pur agile ed ariosa nonostante il gioco complesso dei suoi elementi, si coglie più da vicino quando si pensi alle sue minuscole proporzioni (il lato del quadrato non supera gli otto metri). E' stato detto e giustamente, dal suo primo illustratore, Freshfield, che essa, tra quelle siciliane, è l'unica cui convenga la qualificazione di bizantina . . . ma non è dubbio che la costruzione, per il suo intimo significato architettonico, è già al di fuori della mentalità tardo-romana cioè al di fuori – e si dia al termine la massima estensione – della tradizione classica. (Bottari 1956)

Later on, Charles Nicklies (1994) made an accurate analysis of architectural characters of the Cuba. His study is a crucial point of reference for establishing the church's datation: between the end of the XI century and the beginning of the XII. As a matter of fact, in this period a lot of monasteries were built in the eastern part of Sicily: in 1092 was founded, by permission of Roger I (Re 1996), the monastery of San Salvatore di Placa, whose ruins are still over a rock not far from the cuba (Giglio 1992b). This monastery controlled and managed a lot of fields in the neighbourhood, and had two *metochia* at the moment of its foundation (Re, 1996), two little churches, or chapels, probably with a little house for the monks who received tributes from countrymen. Another metochion related to the monastery was probably the Cuba. As it is a lonely church built far from the town, it completely responds to the function just described.

Aesthetical and constructive characters, stylistic influences

With an analysis of architectural and constructive aspects of the Cuba, is possible to join this building to the particular architecture of the Norman period of Sicily.

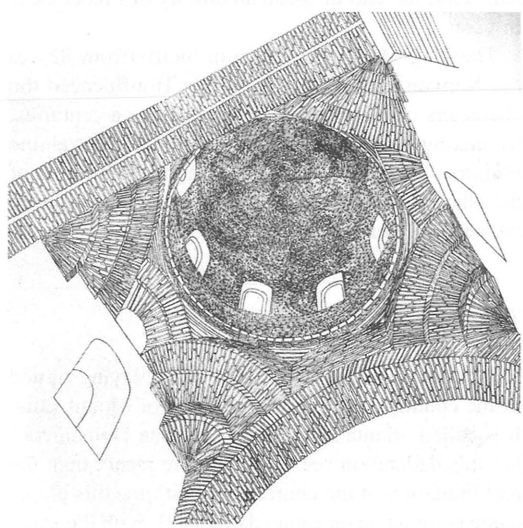

Figure 3
Church of Saints Pietro e Paolo in Agrò, Sicily (1117). One of the two main domes, both built over arches that connect them with the walls. (Basile 1975)

The church concentrates numerous elements, in plan and in the spatial characters of the interior. The square plan (9 meters for each side) is defined by three bays attached to the transept and an apse, the only element protuding out of the linear outlines, scanned by buttresses on three sides, and other two around the apse. One can enter the building by a large door with rounded-head arch, or by a smaller one connected to the nort aisle, both located in the west façade, where a triple-light window flanked by two little ones; two couples of windows bring light inside from south and north wall, while a double-light window is opened in the apse wall. The exterior is lacking in decoration, except for trichromate arched lintle over the main windows and some simple ones.

The interior is very impressive because of the variety of vaults, different for each space they cover: from the main entrance one can see the large, squared central bay, covered with a domical vault. This space is flanked by the narrow side-aisles, with their sets of three cross vaults supported by corbels. The three bays are separated by round-headed arches from the transept, composed by the bema covered by a cross-vault, and two side spaces (*prothesis* and *diaconicon*) barrel-vaulted, where two niches are contained within the thickness of walls.

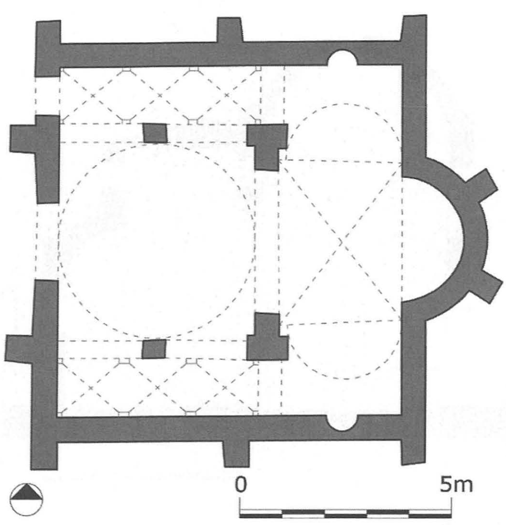

Figure 4
Plan of the Cuba. (Survey by authors, directed by prof. Gennaro Tampone)

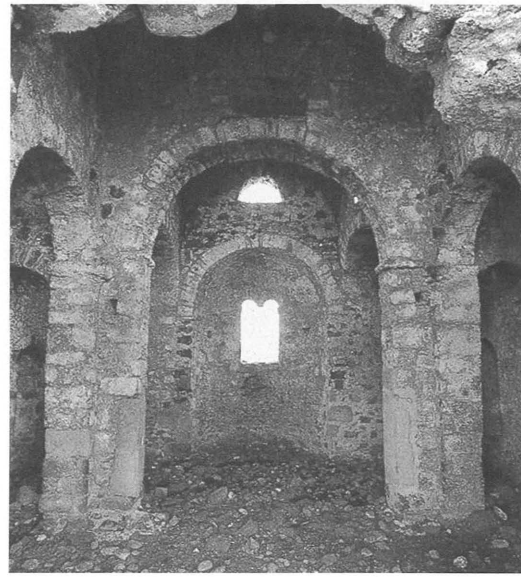

Figure 5
Interior view of the Cuba

As we can infer from this description, the Cuba is a mixture between the basilical type and the centric one: because of the shortness of the aisles and because of the square shape of the central one that is prevailing over the two others, in plan and in height. The vault of naos draws visitors' attention more than other elements of the building, and this maybe was the aim of the planners and the builders. The construction of the vault will be explained later; now is necessary to examine its shape and its strategical position in the equilibrium of the whole building, also because of the uncertain datation.

Actually we cannot find examples in Sicily similar to such a vertically-strained shape, but in Puglia there are a lot of so-called churches «with domes on axis» (Messina and Dell'Aquila 1998), with shapes of dome vaults analogous to the Cuba's one.

These similarities are interesting because they can be temporally linked with the great migration of the monks from the East to Puglia, and then to Calabria and Sicily. In 1059 the Norman Robert the «Guiscardo» became Duke of Puglia and Calabria, after the alliance with

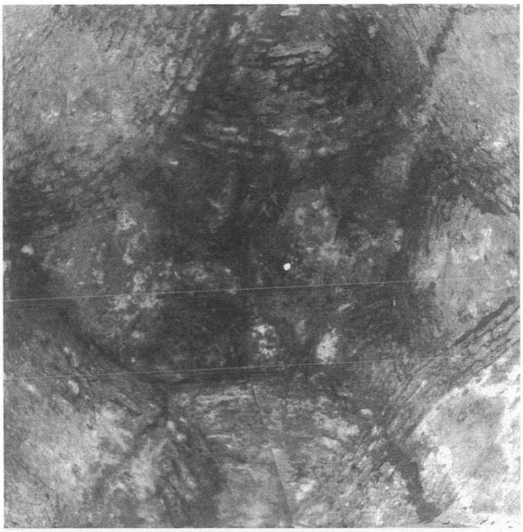

Figure 6
The Cuba. View of the dome-vault over the naos. (photograph by authors)

Rome's Church, and subjected many Byzantine communities, while introducing the latin religion. The monks, who arrived in these regions from the East various decades or centuries before, were then forced to move towards Calabria and Sicily, by following the Normans. In reality, the new sovereigns were not interested in establishing a unique religion for the latest acquired territories. On the contrary, they used to spread both the two Christian worships, from Rome and Byzantium, depending on the community's customs they found over their path. However, a lot of Byzantine monks, allured by the conquest of new lands over the Islamic *oppression*, accepted to move while carrying their experiences, also concerning architectural issues (Scaduto 1947).

This is a key by which we can explain part of the influences that may be found in medieval architecture of Southern Italy, expecially of Sicily and of the Cuba too, where significant experiences from Greece, Armenia, Turkey and Puglia have been mixed with Islamic characters.

THE DOME-VAULT: CHARACTERS AND CONSTRUCTION TECHNIQUES

The stone used for all the Cuba's valuts is pumice-stone (specific weight 0,001 kg/cm^3) from Muont Etna, while the walls and the arches are made of basaltic stone, heavier about two times and a half than the pumice. This difference helps in engaging with less weight the supporting structures; moreover the vertically-strained geometry of the dome-vault distributes thrusts over the wall section below (thickness of about 60 cm) in an effective way, making the resultant closer to the core of the section. The eccentricity of the resultant over the western wall (today the most damaged) is *de facto* acceptable, and the serious damages we can see in this part of the building are due to other reasons.[3]

The little stones, by which the intrados of the dome-vault is made, are similar to bricks in their shape (about $8 \times 12 \times 20$ cm); the peculiarity of this structure is the bond by which these stones were laid, in a special way where no centering was required, at least for the first phases of the construction.

The section of the dome-vault, with a thickness of about 40 cm, is composed by an intrados with regular hewn stones for about one third of the section, and by a second cover over the first, made of rough stones,

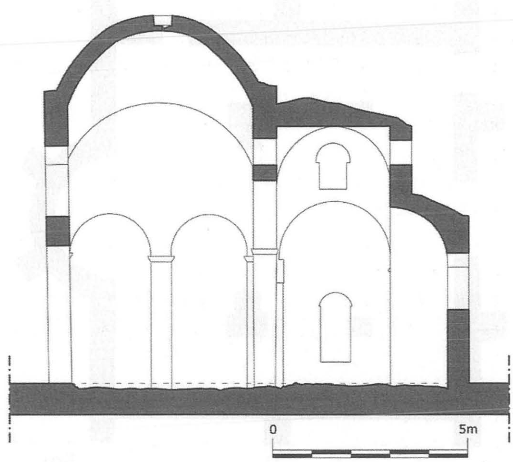

Figure 7
The Cuba. Longitudinal section: the vertically-strained shape of the dome vault is evident here. (Drawing by authors)

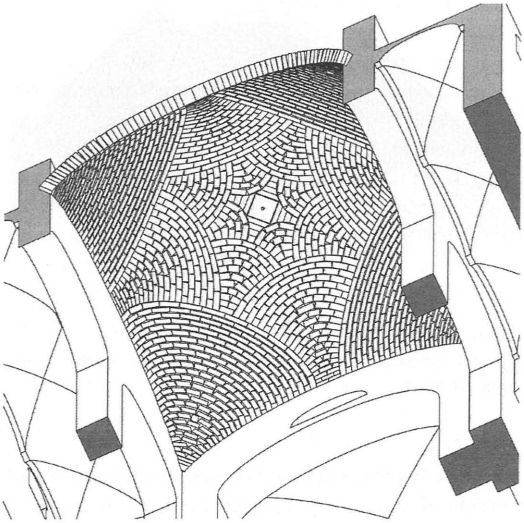

Figure 8
The Cuba. Virtual reconstruction of the squinches's sets. (Drawing by authors)

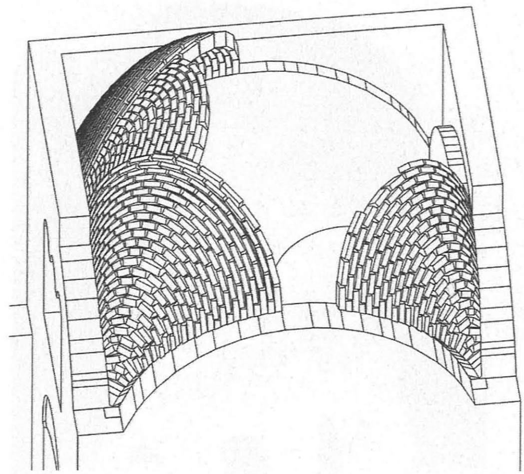

Figure 9
The Cuba. Virtual reconstruction of the constructive process. (Drawing by authors)

over which the roofing tiles were laid.

The construction of the vault began when the basaltic stone walls were raised until the shoulders of the inner shell: over one side of the wall was a level for the regular hewn-stones (creating an arch on the inner faces of the four sides of the naos), on the contrary the external side of the wall was built entirely with a square line-drawing on the four side of the church.

Over the rounded-level left in the wall, the builders laid the first projecting stones in the four corners of the central bay, making a basis for the first squared stones, arranged in arches laid one upon the other. According to this, the projection of the arches in plane is turned of 45 degrees respect to the wall perimeter.

When the four angular squinches were completed, the gap between them and the external walls was filled with stones and mortar. Later, between the four great squinches, other four squinches were raised, turned of 45 degrees respect to the first ones and so ensuing parallel to the walls. Builders used a fairly quick-setting mortar, that let them to build the first three or four sets of squinches without centering. Furthermore, the inner shell of the brickwork is basically a centering for the outer one, that was begun after the second set of squinches was laid.

The other sets were embedded one upon the other, smaller and always turned of 45 degrees toward the keystone. It is a squared wedge stone with a hole, squared at the same way, maybe useful to put inside an iron cross.

The only necessary centering was probably the one needed for sustaining the last two sets of squinches, and for the keystone. The holes in the perimeter walls of the central bay could receive the scaffolding structure for this little centering.

The origin of the construction techniques

In some ways, the structure described above is similar to some famous buildings, very far in space and in time from the Sicilian countryside of the XI–XII centuries. However, it may be useful to run shortly over the history of this construction technique by following an evolution begun with the Diocletian Mausoleum in Split (ended in 302). Here the arches, made of bricks, compose the spherical triangles that create the great self-supporting vault. But the

Figure 10
Ardashir's Palace in Firuzabad (224–242). The ruined vault built over pendentives. (Sanpaolesi 1978)

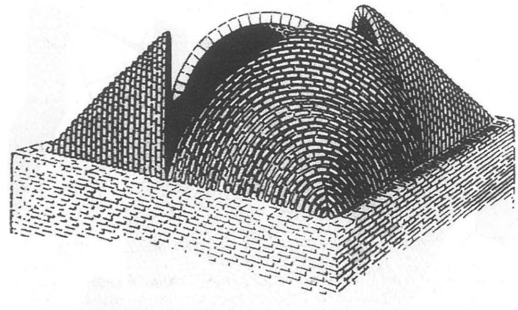

Figure 11
Typical constructive process of a Persian vault. (Pope 1938–1939)

technique used in this *roman* building «non viene da Roma, non viene dalla Siria che usa la pietra, ciò deriva dal procedimento persiano-mesopotamico» (Monneret de Villard 1915).

Therefore, the origin of this kind of arranging vaults is to be found from Persia, where Sassanian skilled workers rose structures as the dome of Ardashir's Palace in Firuzabad. This imposing brickwork structure is connected to the walls by «quattro archi in pietra diagonali sugli angoli» (Sanpaolesi, 1978), four pendentives. It is one of the earliest examples we can find in the history of architecture: this ancient technique is the basis of a practice more and more commonly used in the most representative Persian buildings, as the dome in Sarvistan (V century) shows, according to the sketch reported by Arthur U. Pope (1938–1939).

During the following centuries the Muslims invade Turkey and besiege Byzantium: these events influence the architectural techniques, as many examples in Greece, Turkey and the capital Byzantium can show.

Indeed, the Persian constructive practice become very common. It covers little squared spaces with domes or vaults, arranged with brick's rows, and makes geometrical drawings based on the principle of rotation of squares by 45 degrees, that is a *mandala* with a symbolic meaning: the place where earth meets the divine has always to be emphasized.

The example closer to the dome-vault of the Cuba is the rich group of vaults of the more ancient part of the Friday Mosque in Esfahan. Here a great number of spaces, shaped in square or rectangular and defined by ogive arches, are covered by vaults in which the

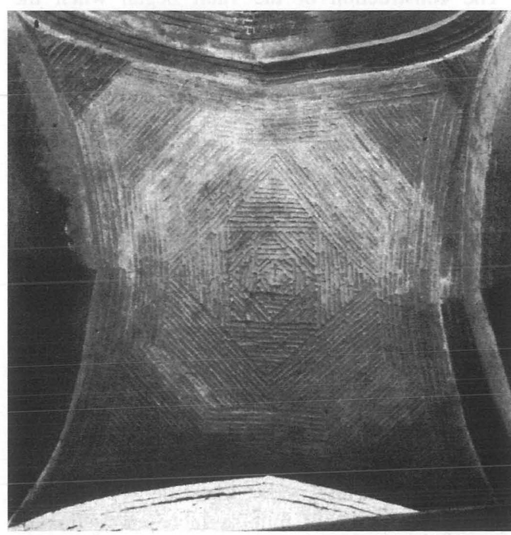

Figure 12
Friday Mosque in Esfahan. One of the vault built in the X–XII centuries. The process of rotated squares is evident. (Galdieri 1973)

same principle of rotated-squares applies. The technique is comparable with the one described for the Cuba's dome-vault: the first squinches are laid over the corners, and the space between them is filled by other squinches smaller and smaller. The key is made by a spiral of four bricks or by special wedges, as four triangular bricks (Galdieri 1973).

In Persia this technique had a wide spread, thank to the great skill of the builders, and to the possibility to cover little spaces (rooms with the side of 5 or 6 meters). It becomes a real trademark for the madieval Persia. During the X and XI century, this practice became very common also in the Byzantine Empire, when a drastic reduction of the spaces occurred in churches' construction, because of the crisis of the Empire. The magnificent domes are now replaced by smaller and domestic spaces, often covered by barrel or dome vaults: in this period we can find a lot of meaningful examples with the same constructive conception of rotated-squares, applied in different ways depending on the various aesthetical and constructive requirements. These examples form a complete collection, together with the Friday Mosque, of the ways in which you can cover a square place with a vault made of bricks.

Sometimes the geometry is misshaped in order to avoid use of centering, and builders tried to find always a new way to do it: the squashed vaults in Saint Panteleimon in Thessaloniki are in contrast with the vertically-strained ones in Istanbul, at Panthocrator, but derive both from the same constructive will (Ousterhout, 1999).

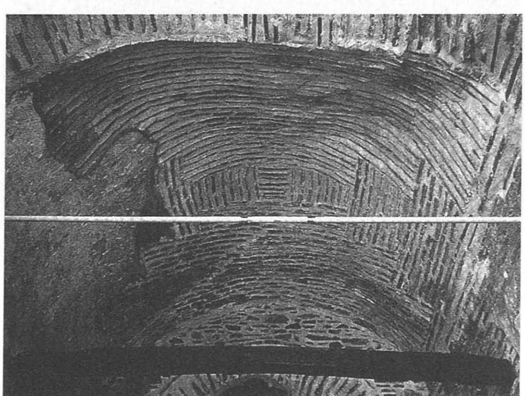

Figure 13
Saint Panteleimon church in Thessaloniki (XI century). A lateral chapel's vault. (Ousterhout 1999)

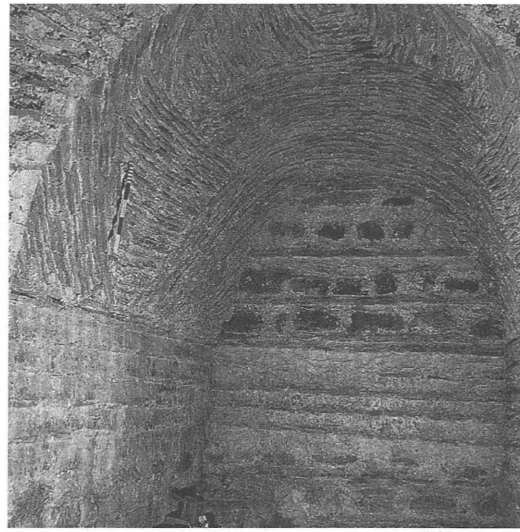

Figure 14
Panthocrator church in Istanbul (XII century). Crypt's vault. (Ousterhout 1999)

If these vaults are arranged in a rough way, somewhere else the same technique creates so much rich and precious drawings that the builders didn't cover the arrangement with plaster. For instance, in the crypt of Saint Demetrius in Thessaloniki (X century) the vaults are somehow still connected with the roman practice: the bricks form, on the vaults' surfaces, unloading and following arches, that do not connect the sides of corners with squinches, but they extend them with parallel rows making fanlights intersecting on the diagonals of the vaults. On the contrary, the semi-domes of the apses at Saint Aberkios in Kursunlu and Saint John in Trullo in Istanbul (XII), show *in toto* the persian influences: the spherical triangles are laid one upon the other (Ousterhout 1999).

Structure and decoration: covering as celebration of construction

As most of the vaults' examples described above show, the function accomplished by the vaults (covering little spaces) adapts itself to the

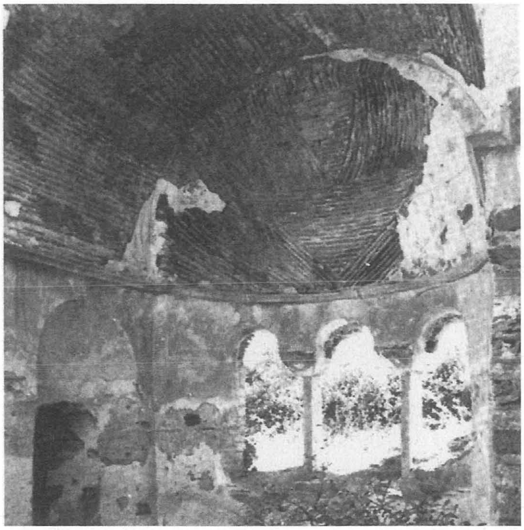

Figure 15
Saint Aberkios church in Kursunlu (XI century). The apse's semi-dome is made of squinches conceived as spherical triangles. (Ousterhout 1999)

requirements of the Byzantine churchs of the late period; but in this way they end their role: a mere constructive role.

The case of the Friday Mosque in Esfahan is different; here some of the vaults have plaster over their brickwork, painted by following the arrangement of the brick's rows, or sometimes by *inventing* new ones. Decorations emphasizes the masonry, both following the brick's arrangement and denying it: in some examples, where the plaster is still surviving, one can see paintings that seems to repeat exactly the brick's rows, even if with a bigger size and, obviously, with less rows in the number. Paintings represent a feasible structure, with dark coloured bricks and white mortar filling. On the contrary, in some cases the vaults' plaster was left white and, over it, only thin lines were painted (fillings between short sides of bricks are missing). It symbolises a different decorative value: covering takes a step forward, toward an abstractionism typical of the Islamic decorations, that will soon leave the structure behind and will approach muqarnas (since this period more and more three-dimensional and independent from masonry).[4]

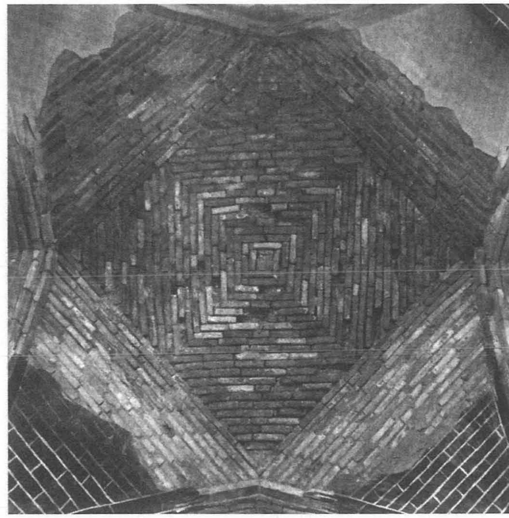

Figure 16
Friday Mosque in Esfahan. Over this vault, paintings reproduce bricks. (Galdieri 1973)

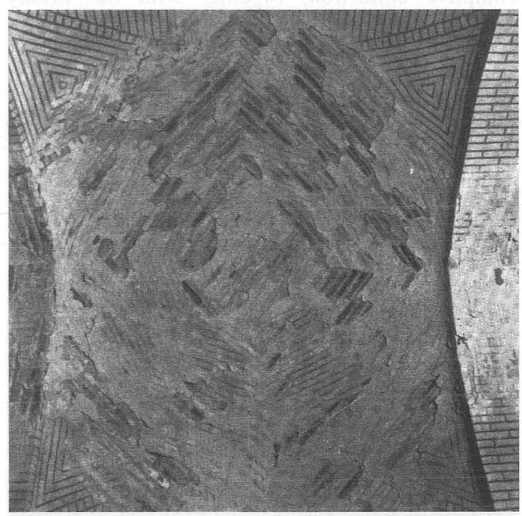

Figure 17
Friday Mosque in Esfahan. In this case, over the plaster only rows are represented. (Galdieri 1973)

The practice of reproducing structures over plaster, being over many vaults in the more ancient wing of the Friday Mosque, was recently also recognized in the Cuba near Castiglione di Sicilia.

Indeed, thank to the repairs, conducted by Soprintendenza ai Beni Artistici e Storici di Catania in 2000–2001, a painted plaster was discovered over the inner surface of the dome-vault: the whole system of squinches is repeated with red and black rows one upon the other, and white fillings. The distemper is spread over a thin plaster layer, the only one present over the stones. As a result, this decoration has to be considered the original one of the church.

Finally, it is important to point out that who settled such decoration for the vault, gave up using traditional symbols of Byzantine worship: indeed paintings do not represent Panthocrator (that was probably painted on the fresco, now missing, in the semi-dome of the apse), neither a cross in correspondence with the keystone, nor stars over the sky. The painter preferred to represent the masonry, the *brickwork* made by skilled workers, overlaying the two-dimensional arrangement over the three-dimensional one.

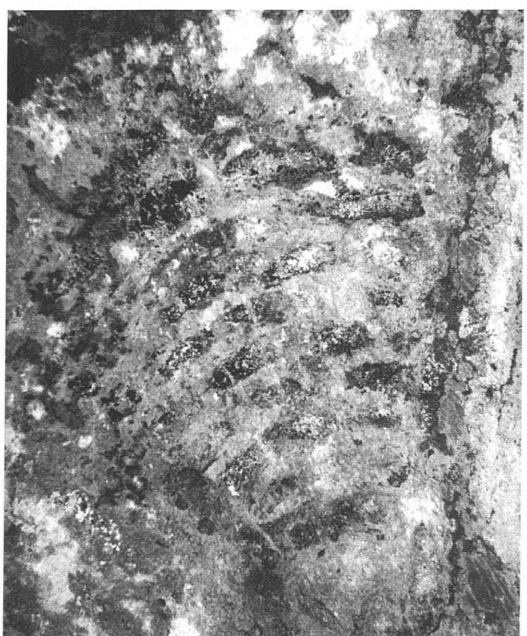

Figure 18
The Cuba. One of the squinches nearest to the corners, during restoration. Under a layer of thick plaster, another layer shows the black and red paintings. (Photograph by authors)

Conclusion

To sum up, because of the evidences reported in this paper, the dome-vault of the Cuba seems to be built by Persian or Islamic skill workers; they chose for the vaults' construction the material easier-to-find in that place, pumice stone, but used it as bricks. For this reason a masonry very close to the eastern experiences was possible.

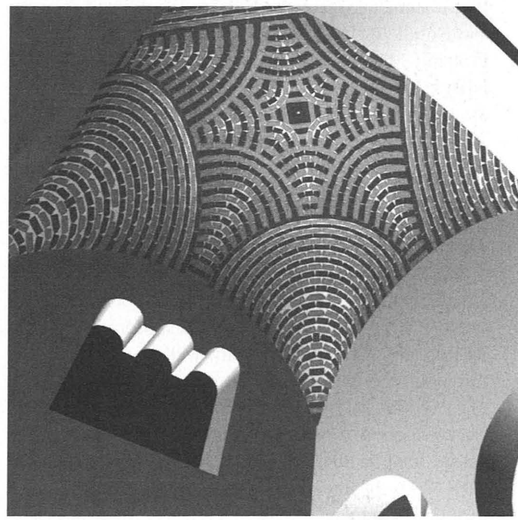

Figure 19
Virtual reconstruction of the main vault of the Cuba. (Drawing by authors)

Islamic influences makes possible to put a *terminus post quem* in 1061 for dating the building: before the Norman conquest, indeed, Muslims did not let any building of churches in Sicily. The Cuba could be built duribg the 30-50 years when a lot of monasteries were risen up, especially in the eastern part of the island.

The Persian —or Eastern in general— characters can be explained with the migration of monks that occurred in the second half of the XI century from Puglia to Sicily, because of the arrival of Normans in Southern Italy.

Notes

1. The first map of the zone edited by the Istituto Geografico Militare in 1891 (Foglio 262 della Carta d'Italia, Tavoletta IV), reports the place-name «Molini San Domenico»: it could be referred to a building, still existing near the Cuba. The mill is sited on the banks of the river Alcàntara. Moreover, a sketch representing the Cuba, appears over another map, more ancient than the first: it is the map n. 116, edited by the Catasto Borbonico, and it was drawn by architect Vincenzo Musumeci approximately in 1850. This is probably the first document where the Cuba is represented (Caruso and Nobili 2001).
2. Probably, the appellation «Cuba» is derived by the Islamic presence in Castiglione di Sicilia, that survived the Norman conquest: *qubba* means a domed space, in Arabic. However, many little buildings in Sicily are named in this way, not depending on their building's age.
3. The original configuration of the Cuba had certainly a narthex, that provided an effective reaction to the thrusts coming from the dome-vault; unfortunately this structure was dismantled (probably by countrymen) in order to get easily building materials. Moreover a door was opened inside the southern wall of the church: in this point the wall was damaged from the arch of the new door to the dome-vault. The restoration intervention in 1959 repaired this damage, but it is still active and another intervention had to be made in 2001–2002 by the Soprintendenza ai Beni Artistici e Storici di Catania.
4. Charles Nicklies (1994) underlines the similarity between the Cuba's squinches and the muqarnas because he sees the former as forerunners of the latter. The church of Saints Pietro and Paolo in Agrò (founded in 1117, Figure 3), twenty kilometres far from the Cuba, is reported as a middle-step between squinches and muqarnas: only one, certainly, amongst the heterogeneous and rich complex of similar examples datable around the Norman period of Sicily.

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The Grand-Place of Brussels. The XIXth century restoration: Diversity of materials and structures

Paula Cordeiro

The state of Grand-Place houses at the end of the 19th century is the result of two hundred years of tumultuous history.

In 1793 the French sans-culottes removed practically all decorative elements from the façades and from the interior of the houses.

The buildings were sold as national property and they continued to deteriorate as the new owners cared little about preserving the decoration of the façades. Significant modifications like the suppression of the gables and the changing of the level of the floors were carried out.

Around 1850, the state of conservation of the façades was alarming. Pieces in danger of falling were repaired in a bad way or simply removed.

Although the City authorities calls the attention of several owners to the bad condition of their property and the need to start a restoration, only some repaint their façade or renovate decorative elements.

First measures

Under the mandate of the mayor Charles Buls, the services of the City undertake the first systematic restoration campaign on the façades of the houses of the Grand-Place. Negotiations with the owners were started in this direction; a convention was signed between the city and the owners.

The City finances and supervises the restoration of the façades. A tax was levied on the owners, based on the surface area of the façade. From then on it was decided that any changes to the façades would require the agreement of the City authorities.

In order to restore the original appearance of the façades, different types of works were carried out on the façades, from simple restoration to total reconstruction. The works were carried out between 1885 and 1923.

The restoration envisages the renewal of walls, the protection of masonry by applying a plaster, the protection of all horizontal surfaces with zinc, the application of white lead paint on the masonry wall and the re-establishment of the decorative elements corresponding to the state of the 18th century. The reconstruction of the façades used the same principles, but extended to the total replacement of the elements constituting the façade.

In order to define the kind of operation which each concrete case requires, a scaffolding is placed in front of the façades to be restored, from which surveys and readings are taken. The stripping of the plaster carried out during this preliminary work makes it possible to discover the original places of the disappeared ornaments, the mullions, muntins and lintels that were removed, and exposes the lower stones of the façade. This information is indicated in precise measured drawings, while copies are made of the existing damaged decorative elements so that they can be replaced.[1]

Parallel to this archaeological research, the restorers do not hesitate to recreate the decorations,

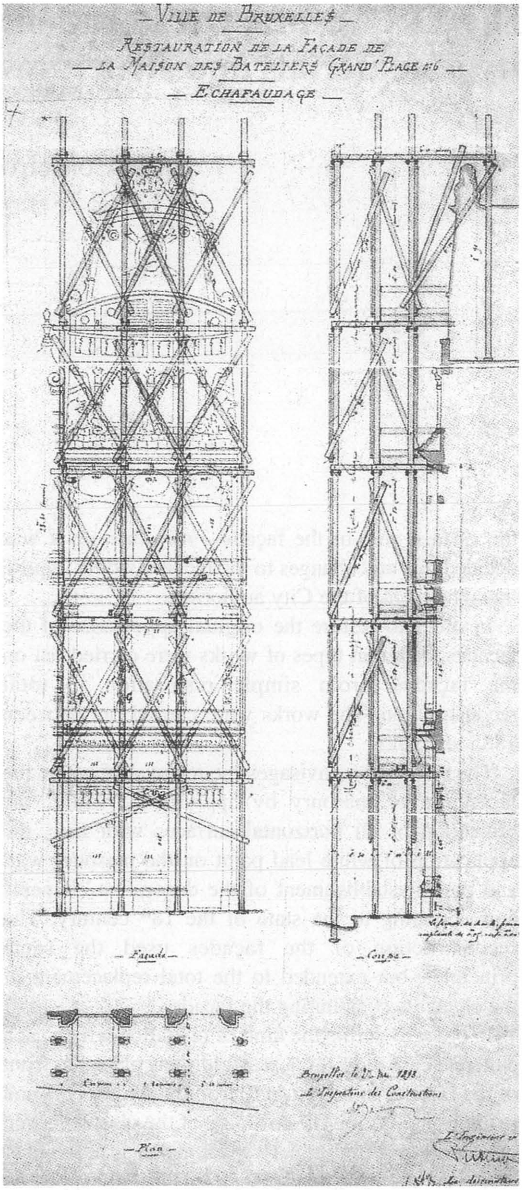


Figure 1
Le Cornet. 6, Grand-Place. Plan of the scaffolding. Scaffoldings are placed in front of the façades to allow the restoring work; they are used to realise the surveys and tests necessary to the elaboration of the technical prescriptions. *Archives de la Ville de Bruxelles PB D5*

which they considered missing, based on the analysis of the written sources and the old iconography documents in their possession.

Through these various approaches, the restorers of the 19th century try to rediscover the aspect of the façades according to the old iconography documents that have been preserved. To make these restorations, they introduce into the old structures irreversible technical changes using modern building materials.

Building materials

A careful analysis of the façades in their current state, combined with the archived study, made it possible to identify the construction materials used by the restorers of the 19th century and to understand how they were placed.[2]

We can observe a significant change in the selection of materials. Mainly made out of Lédien sandstone, the façades were restored in Euville and Gobertange stone.[3] The same happens with the old bricks, which are replaced by modern bricks, generally the klampsteen type. This change is explained by the gradual decline in the use of Lédien sandstone on the stone quarry and by the increasing demand for bricks which require faster manufacture, and the reduction of formats.

For specific repairs, the restorers use products such metal cement, also called Bertagna. This mixture is intended to close hermetically the joints of the stones, to repair the damaged parts and to even mask completely the defects.

In more extensive work, such as the rebuilding of façades or entire houses, metal elements —beams with a I shape, bars of square section, angle bracket, bolts . . . — are integrated into masonry walls, where they serve to reinforce the structure of the building.

The roofs are sometimes covered with tiles, sometimes with slates. For work on this part of the houses, the architects envisage the repair of the zones directly contiguous to the façades. It seems that, at that time, no questions were asked about the authenticity of the type of roof used. In addition, the complete restoration of the roof was not included. The technical prescriptions mention only the maintenance and repair of the roof.

Lead —a flexible metal not subject to corrosion but heavy and not so elastic— is the material usually used

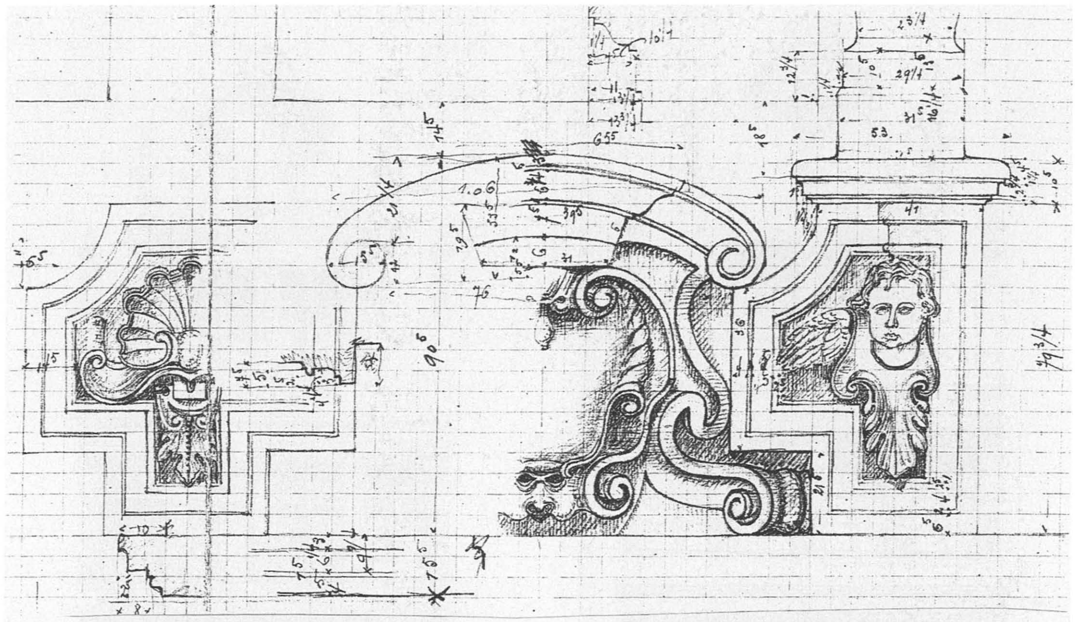


Figure 2
Le Sac. 4, Grand-Place. Composed of many low-relieves and cartridges finely carved, the rich ornamentation of the façade of Le Sac was the object of detailed surveys in 1907. These drawings were made five years before the second restoration of the façade, under the direction of the engineer Jean Segers. *Archives de la Ville de Bruxelles TP 97319*

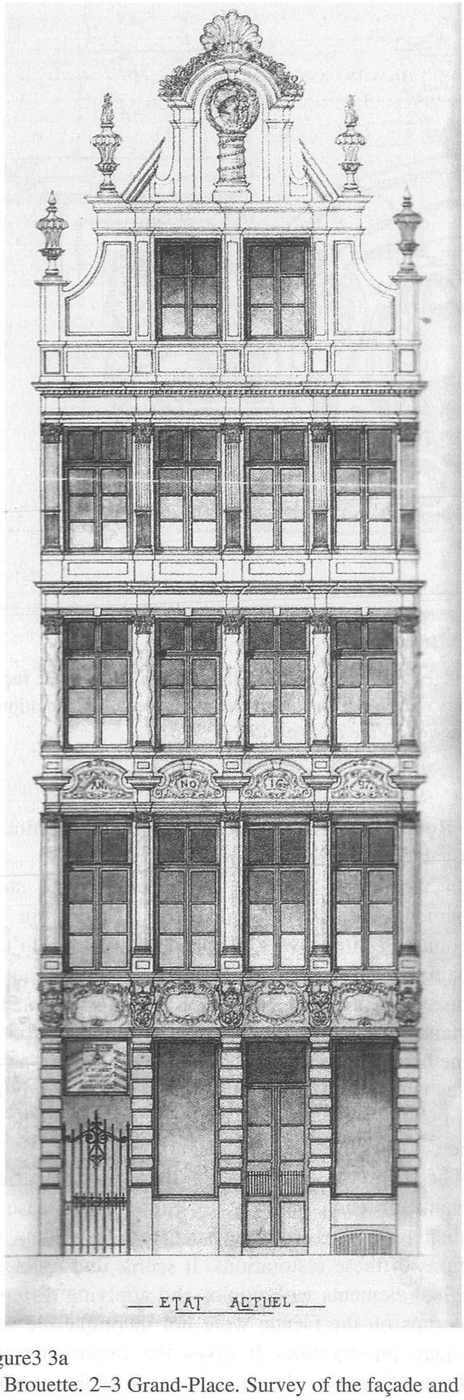
to guarantee the waterproofing of connections between the masonry and the roofs. Gradually, the 19th century restorers will prefer to use zinc, which is lighter, easier to put in work and less expensive. Because of its qualities, it is also retained for the realisation of certain decorative elements such as garlands of flowers and fruits and vases in the restoration of gables.

It's precisely in the restitution of the ornamental elements that the restorers take the most liberties in the choice of materials. We saw that copies were taken of damaged elements. The models were made in terra cotta or plaster and submitted for appraisal by the contracting authority. And although the majority of the decorations were recreated in the stones already mentioned, a number of them were conceived out of zinc, copper, cast iron or bronze.

The fifty-three balusters, which decorated the 2nd and 3rd floor of the façade of the house *Le Sac*, were replaced by masonry before 1850. In 1858, they are replaced in cast iron according to the drawing of F.J. De Rons. At the same time, all the ornamentation of the gable is recreated in zinc.

In the house near by, *La Brouette*, the stone ornaments, which decorated the columns in the second and third levels disappeared and couldn't be repaired with metal cement. They are done out of bronze on a proposal from the architect J Segers. The garlands and the shell which crown the niche of the same house are made out of hammered copper, as the vases which decorate the sides of the gable. Only the statue of *Saint-Gilles*, placed in the axial niche, is made out of white stone of Echaillon.

The main motivation for these modifications remains obscure. Reasons of economy or ease of realising the works are not sufficient to explain the options of these restorations. It seems that replacing original elements with copies and applying different materials on the façade were not incompatible with heritage preservation. It gives the impression that only the form and the visual aspect of the restored element were important for the restorers of the time.



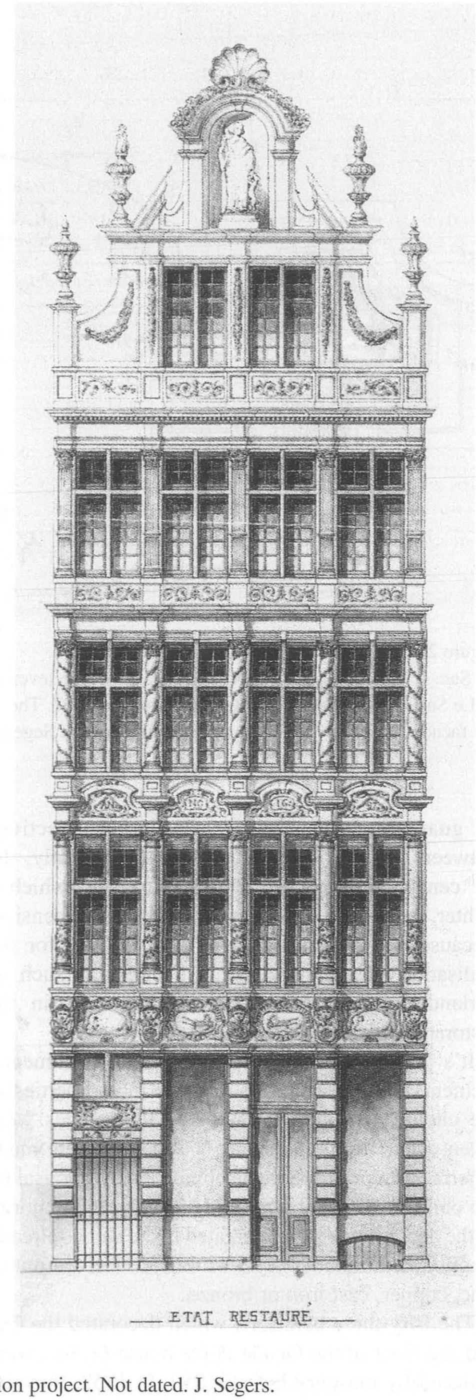


Figure3 3a
La Brouette. 2–3 Grand-Place. Survey of the façade and restoration project. Not dated. J. Segers.
Archives de la Ville de Bruxelles PB D1

Gilding cannot be considered as a material in the strict sense of the term. But it constitutes a main visual element of the decoration of the façades. It underlines the principal sculptured elements and contributes to the aesthetic coherence of the decoration of the square. Judging from what the archives tell us about the state of the façades at the end of the 19th century, it seems that the application of gilding was based on a series of drawings of F.J. De Rons, supplemented by the observations operated in situ at the time of the restorations. In fact, it seems that it also met subjective criteria, guided by the concern of distributing the gilding in an equitable and harmonious way between all the houses.

The techniques of applying the gilding do not change according to the support; only the preparatory layer is adapted to each material. In the specific case of the houses of the Grand-Place, the principal supports are stone and metal. On the cleaned stone, three layers of oil paint were applied, and then the compounding and the paper gilding. As for metal, after stripping, a plaster with a layer of minium lead and white lead is applied and then the compounding and gilding.

Partial restorations

For certain buildings, only a restoration of decorative elements and a repair of masonry have proved to be necessary. The building site of the *Maison des Tailleurs*, started in 1878 with the direction of P.-V. Jamaer, is a nice example of this kind of operation. The result of preliminary surveys leads to the conclusion that the façade did not present structural problems. The operation was limited to the consolidation of masonry with the application of cement plaster and the replacement of the sculpture decoration, on the basis of models carried out beforehand.[4]

A second stage is reached with the restoration of *La Brouette*, *La Louve*, *Le Cornet*, *Le Renard*, *La Rose*, *Le Mont Thabor*, *La Balance*, *Le Pigeon* and *Le Heaume*. In these eight cases, the detailed surveys and tests encourage the restorers to limit the operation to the simple consolidation of the best preserved zones of the façade and to rebuild the most damaged parts, mainly the gables, and those parts were modified in an irreversible way, like the commercial ground floors.

Thus, the project of restoring the main façade of the house *Le Heaume* in 1919 results from this compromise between the rebuilding of certain parts in an identical shape and the conservation of the remainder. The masonry of levels three and four is maintained; on these surfaces, the intervention is limited to the stripping of the colour which covers the wall and the cleaning and the repair of the wall with white stone and *pierre bleue*. On the other hand, the first two levels, the entablature of the fourth and the gable are rebuilt out of Gobertange stone.

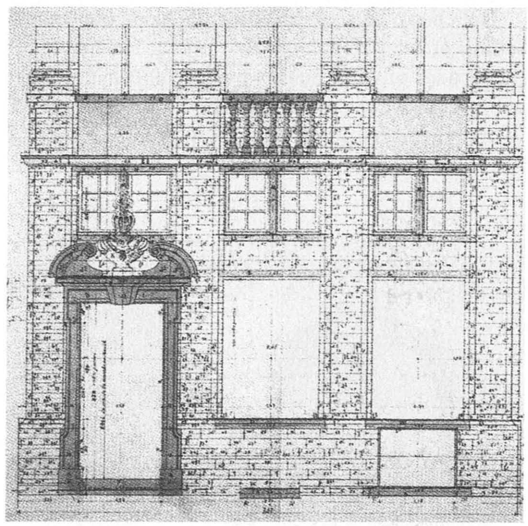


Figure 4
Le Heaume. 34, Grand-Place. Drawing of the levels 1 and 2 for the ordering of the stones. Not dated. The restoration project of the main façade of the house dating back to 1919 envisages the rebuilding of levels 1 and 2 of the entablature of level 4 and the gable on the Gobertange stone. *Archives de la Ville de Bruxelles PB D12*

On the other hand, the sidewall presents a bad state of degradation so that the possibility of rebuilding it is studied. After the coating is stripped in 1920, the old brick masonry, fissured in several places, reveals its weakness. The wall will be entirely rebuilt out of Paepesteen bricks and Gobertange stone, even before the restoration of the main façade.

Carried out in 1897, the simultaneous restoration of the houses *Le Cerf*, *Joseph et Anne*, *L'Ange* and *Aux*

Armes de Brabant illustrates the radical technique of demolishing and rebuilding the façades, realised by the architect A. Samyn. The strongly transformed state of some of the façades does not seem to have constituted a sufficient reason to justify their rebuilding. And there is nothing to suggest that the surveys revealed the need to rebuild for structural reasons. The main motivations of the architect for the decision are not clearly indicated, so we can only make some assumptions in this matter. The desire to rationalise the operations and to conclude the building sites as far as possible probably explain this radical choice.

The choice of such an option of work implies the setting of relatively complex working techniques. If the façades of these houses were demolished in their entirety, all the binders and the floors were preserved. In order to make this operation realisable, scaffoldings were simultaneously placed in front of and behind the façade. In the interior a partition wall was established 0.75 m back from the main façade. This partition wall was built in wood, the interior surface covered in wallpaper and the exterior one in bitumen cardboard. The frames of the doors and windows of the façade were dismounted and replaced in these wall partitions. On each floor, on the lower part of the wall partition a system of zinc pipes is placed to collect rainwater.

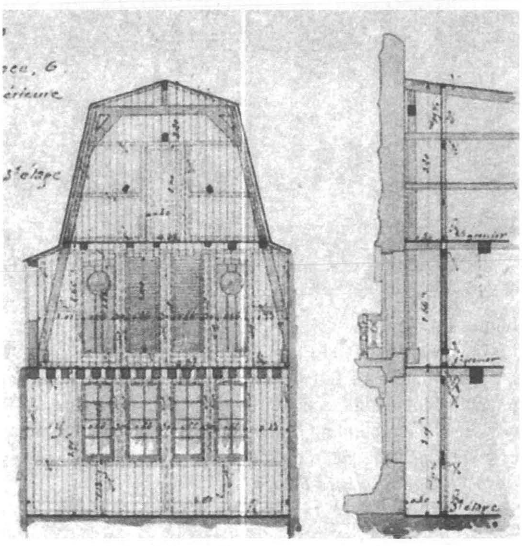


Figure 5
Le Cornet. 6, Grand-Place. Placement of interior partitions on the third floor and the second attic.15 January 1901. The higher part of the façade was rebuilt. Preliminary work consisted of supporting the existing roof and floors and establishing a partition in withdrawal of 0.75m of the façade in order to carry out the disassembling of existing masonry. Frames of the doors and windows were dismounted and placed in withdrawal to form the new partition. *Archives de la Ville de Bruxelles PB D5*

For the re-used cellars, metal beams are embedded in the masonry at the level of the cellar ventilators. This operation is intended to maintain a good connection between the preserved old structures and the new parts. This new element guarantees an adequate distribution of the load of the new façades over all the width of the foundations.

The *pierre blue* is reserved for the thresholds, the steps, the framing of trap doors of cellars, the lintels, the mullions and the support of the reinforcing beams. The blocks of Euville stone of large courses used for the construction of the new façades were bored of holes and seals by nailing. On each level, a steel beam is placed behind the window lintels and the floors plates, while an assemblage of metal beams and angles supports the existing binders. This method makes it possible to intervene only on the façades and guarantees the conservation of all the interior elements of the houses. This system was applied to preserve the privacy of those who continue to live in the house and not to safeguard the archaeological value of these interior spaces.

The specific problem of the buildings on the corner of the block is explained in particular for the restoration of the house *Aux Armes de Brabant*. In this case, the main and side façades were demolished simultaneously in February 1897. The destruction of the side wall showed the very damaged state of the ends of the beams at the height of the roofs, these parts were reinforced with steel angles of 0.12×0.12 m, before being put in the new masonry. The rebuilding implies moreover, the renewal of one part of the interior structures directly contiguous to the masonries, in particular the binders, the floors and the ceilings, the cross walls and the staircase. This façade is rebuilt out of brick and new decorative metal anchors are placed on the façade.

This interventionist method will be applied by the

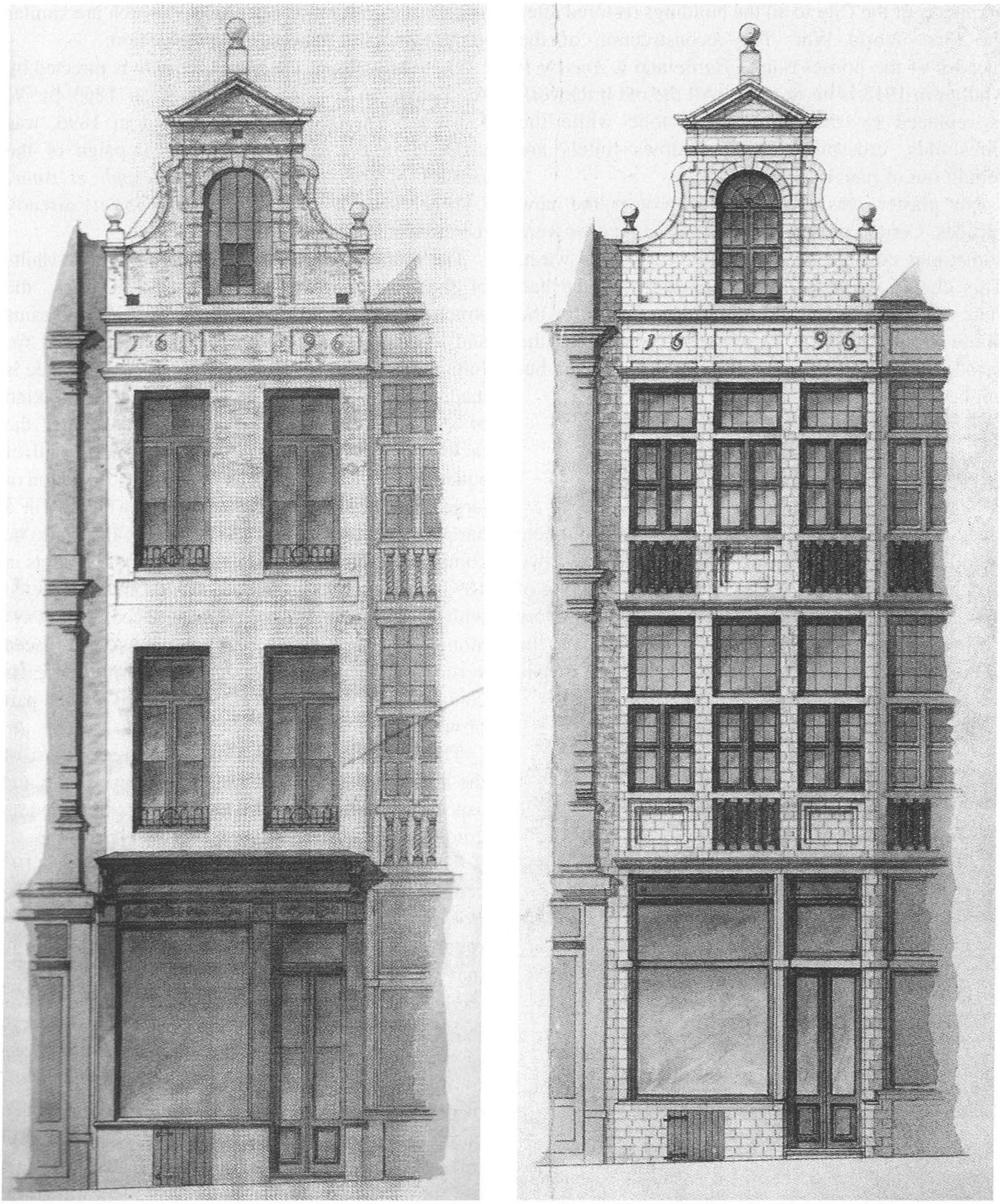


Figure 6–6a
Saint-Barbe. 38, Grand-Place. Survey and restoration project of the façade drawn up by François Malfait, 31 January 1914. This project of Malfait reveals the grandeur of the rebuilding work of the façades. The re-establishment of the original number of spans and the old provision of the ground floor give, in the case of Sainte-Barbe, an idea of the work that was realised. *Archives de la Ville de Bruxelles PB D16*

architects of the City to all the buildings restored after the First World War. The reconstruction of the façades of the houses *Sainte Barbe* and *L'Ane* by F Malfait in 1918 is an example. All the old brickwork is replaced by the Gobertange stone while the thresholds, crosspieces and windows lintels are rebuilt out of *pierre bleue*.

No plaster was envisaged to protect the new façades. Certain preserved and restored façades were sometimes covered with a painting or a lime wash. This choice was probably connected with the bad state of conservation of these façades. Thus, for the house *La Chaloupe d'Or*, a new painting of the façade was recommended by the architect Jamaer but applied only in 1899.[5]

COMPLETE REBUILDING OF HOUSES

Restoration at its most extreme, i.e. the complete rebuilding of houses, can be illustrated by two different building sites —the first is the rebuilding of a small house *L'Etoile* destroyed forty years before during the enlargement of a street and the second the demolition/rebuilding of the house *Le Roi d'Espagne* as part of a restoration operation— which are similar in the choices of materials and realisation.

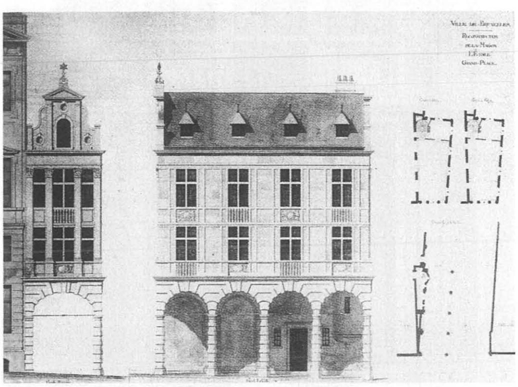


Figure 7
L'Etoile. 8, Grand-Place. Main and side façades and plan of the house. The building was rebuilt respecting the original aspect of the house destroyed in 1853, with the exception of the ground floor. Nevertheless, the materials used are different: the structure consists of iron beams and the façades are entirely in Euville stone. *Archives de la Ville de Bruxelles, Cartulaire de la Grand-Place 55*

The rebuilding of the house *L'Etoile* is directed by A. Samyn on a project conceived in 1863 by W Janssens.[6] This building site, started in 1896, was included in the same rebuilding campaign of the façades of the houses *Le Cerf, Joseph et Anne, L'Ange* and *Aux Armes de Brabant* already mentioned.

The wall masonry of the foundations and the vaults of the porch are built in Klampsteen brick, the structure of the building is made of rolled iron beams and the roof structure is made of Northern red fir. Initially envisaged in Gobertange stone, the façade is made of Euville stone. This replacement of the kind of stone is explained on a technical level by the facility of extraction from the stone quarry and of squaring, the rapidity of delivery and the utilisation of large blocks. Those advantages were welcome in a particular historical context: the rebuilding had to be completed for the Universal Exposition of Brussels in 1897. The stones of the façades are interconnected with staples made of wrought iron placed lengthways along the stone layer and gudgeons are placed vertically. The mortar used for stone masonries is composed of two parts of hydraulic lime for one part of white sand.

The result is a building whose forms recall those of the house destroyed in 1853 but whose structure and aspect betray, to the eyes of the informed observer, an erudite intervention involving partial re-creation.

The construction at the beginning of the 19th century of the building on one corner, known as the *Roi d'Espagne*, is particularly well documented, as regards both the preliminary planning and the management of the building site.

Until the end of the 19th century, this site was occupied by two houses joined behind a unified façade. The state of this façade is the result of several transformations. Ransacked at the time of the passage of the sans-culottes in 1793, its sculpture decoration and inscriptions were destroyed. The modification of interior vertical divisions, probably at the beginning of the 19th century, disturb the architectural composition: the high windows of the first floor were transformed into two superimposed windows and the proportions of those and of the other floors were modified.

The restoration was discussed in 1897, with the

aim of restoring the primitive decoration of the façade. In order to preserve a testimony of it, the existing state was photographed before the execution of the works. Then, scaffoldings were placed in front of the main and side façades. The plaster that covers them was stripped,[7] a survey of the façades was made and the significant elements of decoration are copied.

The result of the examinations of the façades showed that simple restoration could not resolve all the problems of the house. Problems of a technical and economic order will end up leading to its total rebuilding. The first report established in this direction by architect A. Samyn is dated February 1898. It mentions the technical difficulties, in particular for the replacement of the cornices, which would require special support. It also mentions the difficulties related to the erection of scaffoldings that should support the enormous weight of the pillars during the demolition and rebuilding of the subjacent parts. Writing to the mayor in December of the same year, the chief engineer Putzeys mentions several problems: the present state of the façades is the result of successive transformations of the building and this situation seems to him incompatible with a return to the original aspect. Moreover, he affirms that the roof could not support the dome whose restitution is projected. For these reasons, he feels that the destruction of the building is inevitable.

Meanwhile the communal authorities try to come to an agreement with the owners of the two houses, Mr Van den Broeck at No 1 and Mr Vanderton at No 2. They give them an alternative: either they agree to be rehoused elsewhere during the period of the works, or the City proceeds to the expropriation of their property on the grounds of public utility. The owner of No 1 finally yields his house against a thirty year lease for the rebuilt house, on condition of not having to move; the owner of No 2 exchanges his building against two houses belonging to the City and recently built in the *Place de la Liberté*.

But the total demolition of the *Roi d'Espagne* presents some difficulties, because of the outdated structures and the nature of the ground. The work of demolishing the interior reveals that the common walls were in a very bad state of conservation, more serious than the preparatory surveys mentioned. The disintegration of masonry is such as that significant precautions will have to be taken in the realisation of the rebuilding work, the least slip can cause the collapse of the common walls and of the house near by, *La Brouette*. The foundations of the *Roi d'Espagne,* used as a support for the vault that covers the underground of *La Brouette*, are the object of particular care because compressing them could cause the fall of this significant masonry work and the collapse of the whole house. In order to prevent this possibility, the foundations planned for the interior walls are replaced by metal netting beams, embedded in a concrete plate, and the elements that are perpendicular to the gable wall passed under the common walls. The gable wall with two bricks of thickness will double the common walls of all the surrounding houses. The old façades of *Le Roi d'Espagne* will not be demolished before the construction of the new interior walls and of the binders.

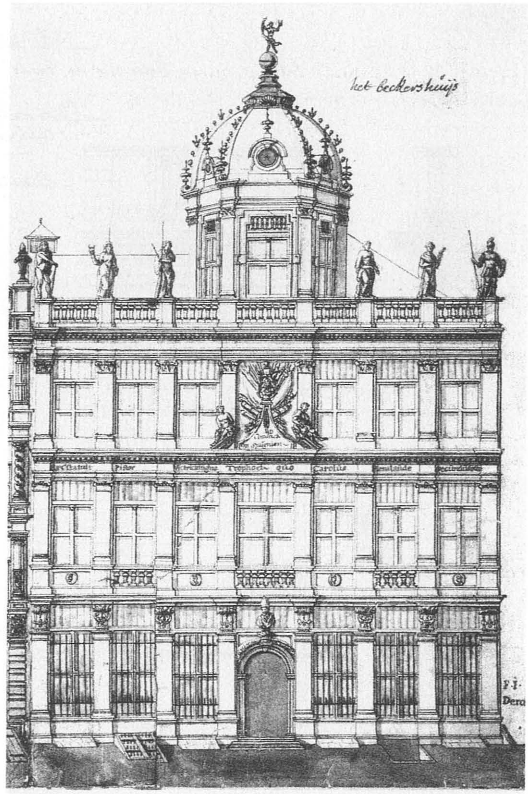


Figure 8
Le Roi d'Espagne. Drawing de F. J. Rons, 1749. *Musée de la Ville de Bruxelles*

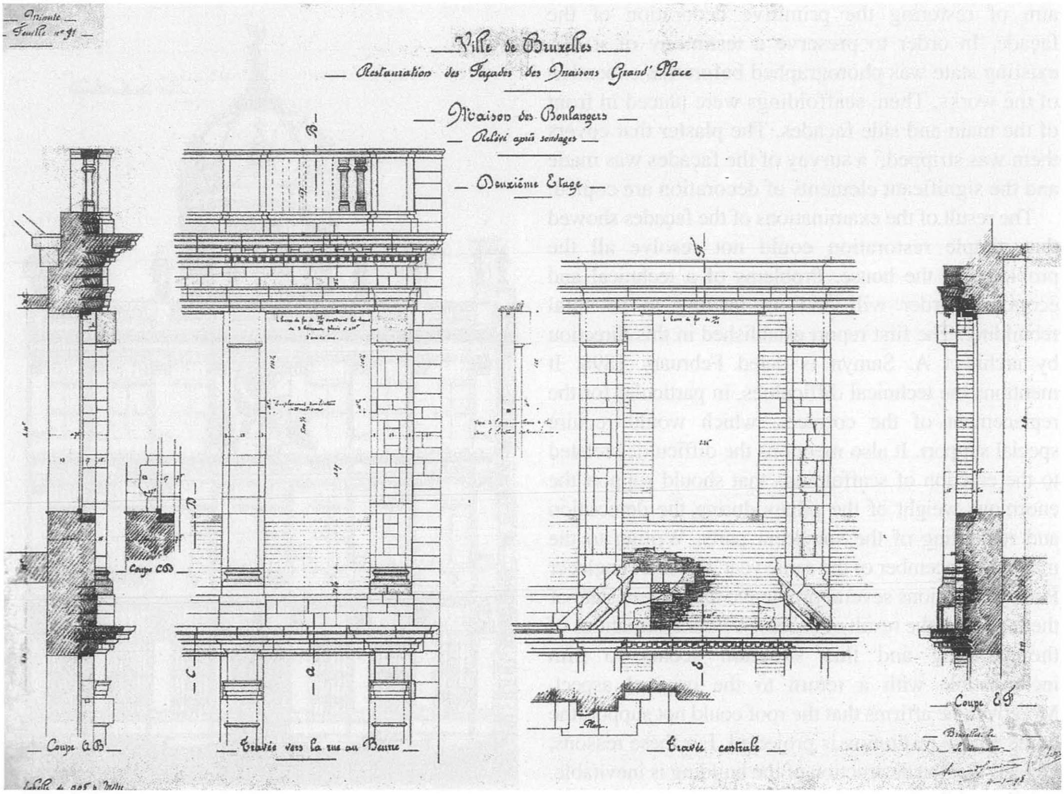


Figure 9
Le Roi d'Espagne. 1, Grand-Place. Survey of the main façade before the demolition of the building in 1900, drawing of A. Fumière, 4 January 1898. *Archives de la Ville de Bruxelles PB D14*

The binder of the ground floor is realised in hollow blocks 0.25 m thick made of cement and gravel concrete of Quenast[8] and that from the upper floors is realised in hollow *kleine* blocks filled out with gravel of Quenast with a total height of 0.20 m. To support these binders, while waiting for the construction of the new façades, metal pillars are established inside the building, against the piers of the existing façades. These pillars still exist, masked behind the interior decoration of the building.[9] All the structure of the new building is in metal, including the floors.[10] Higher than the value recommended from 3 to 5 m, the range of the beams is 6 m while the spacing between the profiles is from 0.65 to 0.70 m —bigger than for the wooden floors— corresponding to the standard used at the time.[11]

The two houses, which will form only one after reconstruction, will be rebuilt in two periods in order to allow the occupant of No 1 to remain in place.

The phases are connected in the following way. The main wall of house No 2 is done up to the existing façade towards the *Grand-Place*; a provisional partition wall is placed on all the floors between No 2 and No 1. Once the rebuilding of the part corresponding to house No 2 completed, the building site of house No 1 is started. This working method means that the new façade is also built in two parts.

The work of demolishing the interior of No 2 starts in July 1900. In November, the foundations are established and the back façade is demolished. The work of rebuilding the interior can start. In 1901, the construction of the foundations of the new façade

started before the dismantling of the existing façade. This very complex management of the operations is well described in the weekly reports of the building site. Thus for the week from 3 to 8 June 1901 it is mentioned that the new façade was built up at 0.90 m of the level of the ground floor while the binder of the attic was already placed. The works in house No 1 will not start before February 1902, according to same logic.

The reports of the building site preserved at the City archives reveal a series of details, which deserve to be mentioned for their complexity and their originality. Thus, all the old beams supporting the truss of the timber frame were initially to disappear but the contractors insisted on maintaining them because of their anchoring in the masonry of the gables of the surrounding buildings. They felt that these beams were a structural connection between the buildings, and their suppression could present risks, so they would have been partially preserved.[12]

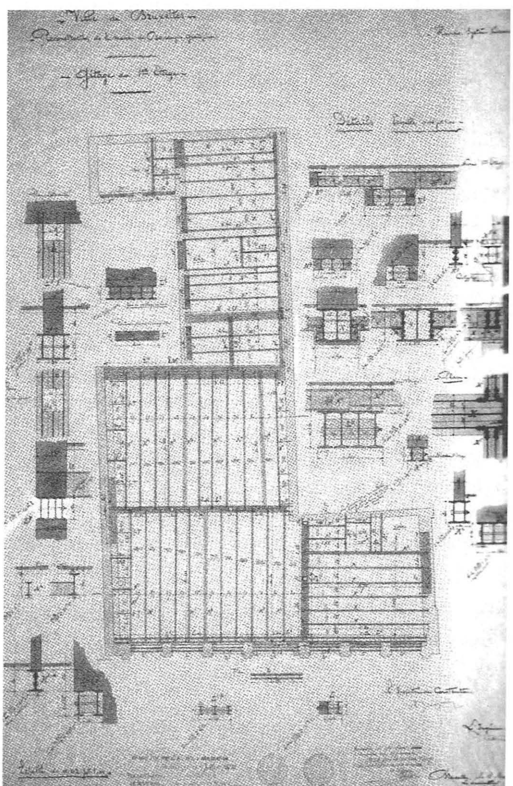


Figure 10
Le Roi d'Espagne. 1, Grand-Place. Plan of the binders of the first level, 15 May 1900. The structure of the floors on the level of the stages is entirely metallic. *Archives de la Ville de Bruxelles PB D14*

The main façade is completely rebuilt in Euville stone. The architect Samyn justifies his choice with four reasons. The first concerns design of the façade: the support points are piers with a restricted section, the best form to place the stones, according to him, is the perpend.[13]

The second is related to the need for rebuilding the building in two phases: the multiplicity of horizontal joints could reveal an unsightly line of connection between the two parts of façade, a problem that the use of a stone offering a great height tends to minimise. The third reason is that the Euville stone allows redressing in place, which could prove to be necessary in order to eliminate the irregularities due to the compressing of construction. The fourth lies in the fact that Euville had already been retained for the sculptured parts of the façade and that the material homogeneity was then guaranteed.[14]

The sidewall is made out of Klampsteen bricks, except for the level of the ground floor, where the Euville stone is used. The back façades in brick are coated with white cement.

Strangely, it is the wish to return to the original aspect of the house *Le Roi d'Espagne* which justified its demolition. But if the means used to realise the works seem to have denied the objectives, namely the conservation of a patrimonial asset in its physical integrity, the reason should be sought in the evolution of the idea of heritage conservation rather than in an error of judgement by the restorers of the time. Once again, it is the aesthetic aspect of the building on which they intervene which worries the specialists charged with the building site, and not any other consideration.

Thus, the visual homogeneity of the Grand-Place hides a great diversity of materials and structures. The programme of restoration of the 19[th] century and the beginning of the 20[th] century had as a consequence the replacement of several façades by new structures and a very heavy intervention on a number of others. The philosophy of these choices was primarily guided by the desire to restore the aesthetic aspect of the façades; priority was given to the restitution of the

decoration elements of all the façades of the square. The preservation of the authentic materials and of the original structures apparently did not form part of the concerns of the restorers of the time. In spite of these reservations, it should be stressed that this vast campaign of restoration returned the splendour to the façades of the houses and the square regained the image of an ensemble that had been lost.

NOTES

This article was published for the first time in: Cordeiro, Paula; Hennaut, Eric; Heymans, Vicent; Lambert, Cécile; Laoureux, Denis; Soenen, Micheline; Vanrie, André. 2001.Les Maisons de la Grand-Place de Bruxelles.*CFC - Editions, collection Lieux de Mémoire.* For the reference list see at the end of the same publication.

1. Examples of very precise surveys of the sculptures decorations from the houses *La Brouette* and *Le Sac* were found in the archives, as well as the photos of the models done for the bust of *Saint-Aubert* from the house *Le Roi d'Espagne* and the statues of the façade of the house *Le Renard.*
2. The notes which will follow are the result of the confrontation of the archive documents with the visual observation of the façades carried out during the inspection campaigns in June and December 2000.
3. The Euville stone is oolitic limestone originally from France, very much used in Belgium during the second half of the 19th century. The Gobertange stone is calcareous sandstone from Brabant, also known as bruxellien sandstone. Both are used in the façades and in the sculptural elements.
4. Invoices for the white stone supply for the new sculpted decorations of the façade (capitals, volutes, and garlands) were found in the archives of the house *La Maison des Tailleurs* (AVB TP 97388).
5. The exact colour of this painting containing white lead or zinc oxide is unknown. Its application was delayed by fear of possible degradation to the cement plaster caused by a very fast application of the colour (AVB TP 97387 and 97388). Beside, the façade of *La Chaloupe d'Or*, the façades of the *Paon*, the *Petit Renard* and the *Chêne* were plastered with a cement roughcast covered with an oil paint (AVB TP 63899). .
6. The description presented in the tender for the rebuilding of the house gives an idea of the type of materials used (AVB TP 8611).
7. The archives do not give any information on the kind of plaster that was applied on the façade.
8. «Cement composition: one part of sand, one part of cement; one part of lime, three parts of gravel of Quenast; coat of 0,10 cm of thickness », report of the enterprise Carsoel that realised the works, 29 January 1901 (AVB TP 57644).
9. Geophysical measures were made in august 2000 to verify the existence of these metal pillars. Grégoire C., Halleux L., Van Balen K. »Mesures géophysiques au Roi d'Espagne, Grand-Place de Bruxelles» Katholieke Universiteit Leuven, 29 août 2000.
10. The metal floor made its appearance in the construction industry during the second half of the 19th century. In 1890, it had already conquered a large share of the market.
11. Technical plans of the floors of *Le Roy d'Espagne* (AVB PB D 14); A.N.A.H, *Les planchers anciens*, Paris, 1979, p. 46.
12. Report of A. Samyn and M. Carsoel, March 22, 1901 (AVB TP 57644). The two front beams (towards the Grand-Place) would have been dismounted while the two posterior beams would have been left in place. Information should be checked at the time of the disassembling of the ceilings of the 2nd stage.
13. Element crossing all the thickness of masonry; thus having two faces in the wall.
14. All the sculpture decoration is made out of Euville stone, except for the bronze statue, which crowns the dome.

From the «*Architecture hydraulique*» to the «*Science des ingénieurs*»: Hydrostatics and Hydrodynamics in the XIXth century

Massimo Corradi

L'ARCHITECTURE HYDRAULIQUE: THE FIRST STUDIES ON RUNNING WATER AND THE FOUNDATION OF HYDRAULICS

Hydraulics, notwithstanding its ancient origins, is very young as a discipline. It has been founding and consolidating its scientific bases only for the last three centuries as pure science, like mechanics, and its application to engineering. The «discovery» of basic principles, the fundamentals of hydraulic science, required many efforts throughout the 17th and 18th century.

The first phase of development is the great season of experimental hydraulics during the Renaissance, especially in Italy, thanks to the contribution of Leonardo da Vinci (1452–1519), Girolamo Cardano (1501–1576), Giovan Battista Benedetti (1530–1590), Bernardino Baldi (1553–1617) and others. Only with the school of Galileo Galilei and his pupils, like Evangelista Torricelli (1608–1647) and Benedetto Castelli (1577–1643), with his treatise *Della misura delle acque correnti* ('On measuring running water') published in 1628, the road to the great treatises on hydraulics of the 17th and 18th centuries was opened. In this field of studies the treatise of Carlo Fontana (1634–1714) «On measuring running water» (Fontana 1696) played a great importance role.

In 1644 Torricelli, an Italian scientist, published in Florence *De motu Aquarum* (Torricelli 1644). In this book, he set the law bearing his name: the first public announcement of his water efflux principle. This law stated that the velocity of water efflux from an orifice in the bottom of a tank is proportional to the square of height from the surface of the water to the bottom of the tank. In other words, this velocity is equal to «liquids (velocity) which issue with violence have at the point of issue the same velocity which any heavy body, or any drop of the same liquid, it were to fall from the upper surface of the liquid to the orifice from which it issues» (Rouse and Ince 1963, 62). It is commonly written as $v = \sqrt{2gh}$. This law will be thourougly explained, with the utmost accuracy, by Daniel Bernoulli (1700–1782), in the first half of the 18th century, by means of differential and integral calculus.

In the 17th century the studies on hydraulics were no longer limited to Italy but they spread all over Europe thanks to the work of Simon Stevin (1548–1620), Edmé Mariotte (1620–1684) —who is considered the father of the experimental method— Marin Mersenne (1588–1648), Blaise Pascal (1623–1662) —who was the most important scientist of the century in hydraulic science— and also Isaac Newton (1642–1727) with his studies on fluid mechanics and, on a more experimental level, Pierre Varignon (1654–1722) on motion and the measurement of running water.

Mariotte's treatise on running waters and fluid bodies (Mariotte 1686) —published two years after his death— led the research on fluid and liquid properties, on the equilibrium of heavy fluid bodies,


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Figure 1
Carlo Fontana: *Trattato dell'acque correnti* (1696)

even compressible, on the measurement of running water, on the trajectory of liquid particles, on water's distribution and finally on the resistance of water pipes under water pressure. Mariotte started from Torricelli's efflux principle and he verified his theories in several experiments by means of ingenious mechanism.

Concurrently, in Italy, Domenico Guglielmini (1655–1710), and considered the founder of Italian school of hydraulics, published very important treatises on hydraulics, the measurement of running water (Guglielmini 1690), and the motion of water rivers (Guglielmini 1697). In this last field, Guglielmini takes again speculative ideas from Castelli and Torricelli, and somehow he established the basics of fluvial hydraulics. Guglielmini was the first scientist who showed the existence of a uniform state of equilibrium between running water on inclined plane (which increases his velocity) and the riverbed's active resistance.

In the middle of 17th century Blaise Pascal (1632–1662) was the most important scientist in hydraulic sciences, particularly hydrostatics. His treatise on the equilibrium of liquids, published posthumously in 1663, extends Stevin's analysis and experiments. The most notable theory in his book is the concept of constant hydrostatic pressure at the same water depth. In a fluid the hydrostatic pressure is the same in all directions which are starting from the centre of water particle. This important contribution of Pascal bridges dynamics of rigid bodies and fluid dynamics.

Moreover, we remember Newton's fundamental contributions on bodies immersed in fluids or liquidsa

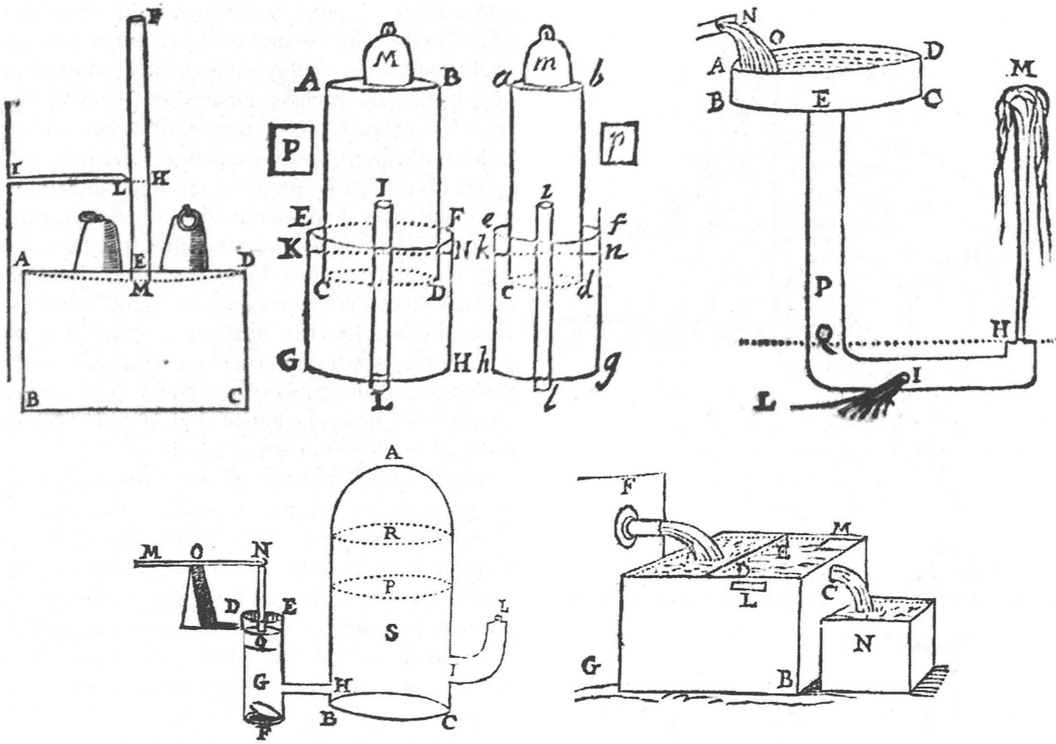

Figure 2
From Mariotte's treatise (1686)

and his studies on water jets, wave-motion, viscosity's coefficients. About viscosity Newton established that tangential stresses in a viscous fluid are proportional to the relative velocity of fluid in adjacent parts. In an endless cylinder rotating on its own axis with constant angular velocity, fluids velocity changes in inverse proprotion to the radial distance measured from the axis. Thus, Newton rejected Descartes' theory on vortex.

In 1725 the treatise of Pierre Varignon on water motion and running water measurement was published posthumous. From this point of view, this treatise focused on the mechanistic aspects of the problem rather than to the fluids behaviour. Varignon analysed the Torricelli's problem: the discharge of a liquid from an orifice. But, as Newton, he obtained the same wrong result about the coefficient which is a velocity multiplier of liquid's efflux.

A few years later, in 1743, Johann Bernoulli (1667–1748) published a treatise entitled *Nouvelle hydraulique*. In this work Bernoulli indicated his point of view of Newton's theory about the shape of a stream of water discharged from a cataract. In this subject Bernoulli called attention to the error in Newton cataract theory, as this hypotheses required a zero pressure —which was physically impossible— throughout the zone of contact between the cataract and the stagnant water around it.

We have to remember though that for the whole of the 16th, and part of the 17th century as well, hydraulics has been confined to the empirical sciences. It was only with the coming of differential and integral calculus, in the 17th century, that the principles of the motion of fluids were established and hydraulics raised to the same level of the other mechanical sciences.

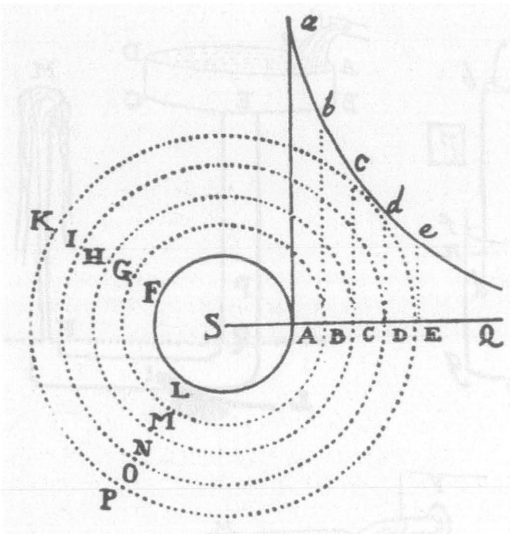

Figure 3
Velocity distribution around a rotating cylinder by Newton

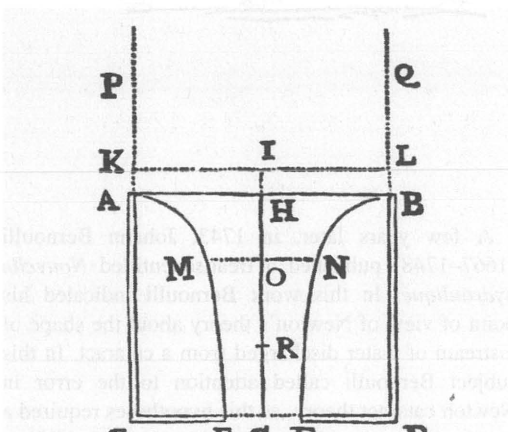

Figure 4
Newton's original concept of orifice discharge

2. The beginning of theoretic hydrodynamics

The beginning of theoretic hydrodynamics dates back to 1738, Daniel Bernoulli published the treatise on hydrodynamics (see: *Hydrodynamica*, 1738). The author gives a simple explanation of the problems related to the static balance of fluids, efflux velocity, liquids oscillation, energy saving principle or energy loss, hydraulics machine, air motion, fluids motion etc. This *compendium* of Bernoulli studies showed Bernoulli concern with theoretical principles and application of the progress in hydraulics, and particularly in hydrostatics and hydrodynamics aspects. For example, we remember that Bernoulli was the first scientist to use the piezometer to calculate the water pipes pressure. This work is a cornerstones of modern hydraulics, in that it is the definition of the fundamental relationship between the speed of an element in a liquid mass and its relative load. In this important work Daniel Bernoulli defined the basis of modern hydraulics.

What is significant is the demonstration of his theorem, based on the energetic principles by Christiaan Huygens (1629–1695) and Gottfried Wilhelm Leibniz (1646–1716), which establishes that in a perfect liquid, in stationery motion, the sum of position, pressure and kinetic energy of each particle is constant during the whole trajectory, which confirms the principle of energy conservation.

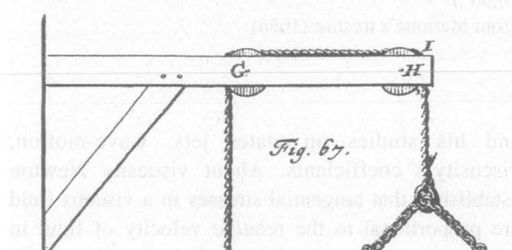

Figure 5
Water efflux from an orifice by Bernoulli

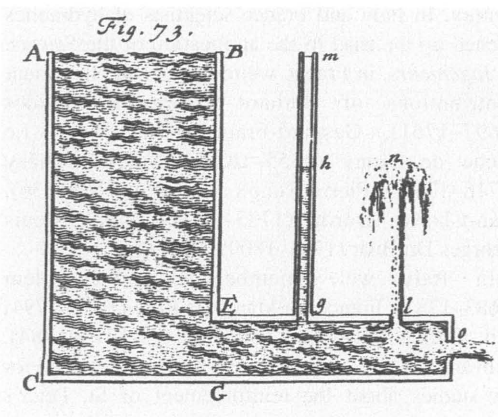

Figure 6
The first idea of the piezometer by Daniel Bernoulli

In formula: $\zeta + \frac{p}{\gamma} + \frac{V^2}{2g} = const.$ (*Bernoulli's Theorem*), where ζ is the height of generic water particle on his trajectory; p is the liquid pressure; γ is the specific weight of liquid; V is the water velocity; g is the acceleration due to gravity

Bernoulli's work opened up the road to important developments in hydraulics by scientist like Jean-Baptiste Le Rond d'Alembert (1717–1783), with his studies on hydrodynamics and fluid mechanics, Leonhard Euler (1707–1783), Pierre-Simon, Marquis de Laplace (1749–1827), Alexis-Claude Clairaut (1713–1765) and many others.

During few years numerous treatise had been published and this scientific literature granted to this discipline the role of mechanics science instead of empirical art. For this reason we make a concise compendium of most significant researches.

In 1744 d'Alembert published his *Traité de l'equilibre et du mouvement des fluides* on hydrodynamics and fluid mechanics. In the opinion of this French scientist hydrodynamics (and then hydraulics) must be founded on experimental observations; conversely, solid mechanics can be founded on the basis of metaphysical principles. The problem related to the fluid motion and fluid resistance were complex; they can be misinterpreted by the *Philosophes à notions incomplettes* —as stated by Leibniz—, but also the scientists with a propensity to give a philosophical halo with metaphysical peculiarity to mechanics principles used in this particular field. The problem —certainly a complicated one— dated back to the problem of *conservation of live forces* which Daniel Bernoulli assumed as principle although d'Alembert deduced it by his «principle».

Three years later, in 1747, the Berlin Academy set a prize competition on this topic and the winner was d'Alembert with his essay *Réflexions sur la cause générale des vents*. This results was strongly criticised by Daniel Bernoulli: he named the winner a «good mathematician» but a «very poor physicist». In fact, the prize was assigned with merit: the d'Alembert's work doesn't solve thourougly the mathematical problem, but in this work was showed important results introducing aerodynamics studies. Also in 1750 the Berlin Academy promoted another prize competition more related to the subject of hydrodynamics and the theory of fluid resistance, but no memoirs was deserved the prize.

In 1752 d'Alembert published in Paris his fundamental work *Essai d'une nouvelle théorie de la résistance des fluides* where he introduced his hydrodynamics «paradox» (or «d'Alembert's paradox»). This paradox stated that a body moving in a perfect fluid or if it is possible to state a related motion between fluid and body, the resultant of the whole fluid pressure acting on body is equal to zero. This result, in conflict with experience, depends on the hypotheses of a fluid without adhesion and viscosity and that the body is acting through an ideal homogeneous weightless fluid.

The next step was the publication of Euler's *Principia* (*Principia motus fluidorum*) in 1755; this work is the benchmark for all scientists of fluid mechanics and hydrodynamics. In this fundamental book Euler defines his *equation of continuity* for a fluid. This equation translates in mathematical form the physical principle of mass conservation, by using potential functions for an incompressible fluid. By using this equation it is possible obtain the equation describing the motion of a fluid, even if —as Euler himself remarked— «it still impossible to have a complete knowledge of fluid motion not for inadequacy of principles, but for lack of instruments in mathematical analysis » (Euler 1755).

There is a great difficulty: it is the mathematical integration of differential equations to partial derivative which describes the motion of generic

fluid, as later remarked by Joseph-Louis Lagrange (1736–1813) in his *Méchanique analytique* (Lagrange 1788, 436]. Finally, it is important to remember Laplace' contribution. Pierre Simon Laplace (1749–1827) introduced his Laplacian operator —in his *Mécanique céleste* published in five volumes starting from 1799 to 1825— to study problems related to hydrodynamics. His studies on wave-motion and tides, in addition to those on capillarity, are very important to develop hydrodynamics. Considering applicatory point of view, Franz Joseph von Gerstner (1756–1832) increased these studies. He also studied hydraulic machines, dams, water motion in channels and wave-motion [Gerstner, 1788].

To synthetize: the indefinite equations of equilibrium in a generic fluid are: $kF = \mathrm{grad}(p)$, where F is the vector force referred to the mass unit, k is the liquid density, p the specific pressure related to area unit. If the liquid is incompressible, then it is possible write that $p = f(k)$. The equations of fluid motion, obtained from d'Alembert' principle, are: $k\left(F - \frac{\mathrm{d}v}{\mathrm{d}t}\right) = \mathrm{grad}(p)$, $p = f(k)$, $\frac{\partial k}{\partial t} + \mathrm{div}(kv) = 0$. The last equation is named *continuity equation*, and it assumes the next form $\frac{\partial k}{\partial t} + k\,\mathrm{div}(v) = 0$ where v is the velocity of a single fluid particle P where the vector force is applied F, p and are respectively the pressure and density in the same point at the time t.

To establish kinetic state of a fluid, or the velocity assumed by fluid particles when through a determinated fixed point in the fluid mass, it is possible to write that: $\frac{\mathrm{d}v}{\mathrm{d}t} = \frac{\partial v}{\partial t} + \left(\frac{\mathrm{d}v}{\mathrm{d}P}\right) v$. Then is easy to have that $\frac{\partial v}{\partial t} + \left(\frac{\mathrm{d}v}{\mathrm{d}P}\right) v = F - \frac{\mathrm{grad}(p)}{k}$. This equation shows the *absolute form* of Euler' equation (Marcolongo 1904).

3. Applied hydraulics

The 18th and the 19th centuries were also the centuries of experimental hydraulics. Experimenters played a crucial role in the development of applied hydraulics. The aim of this essay is follow the same route that leaded from theoretical hydrostatics and hydrodynamics to applied hydraulics during the 19th century. In Italy and France scientists of hydraulics opened up the road to the application of the *Science de Ingénieurs*. in France we remember the important contributions of Bernard Forest de Belidor (1697–1761), Gaspard-François-Clair-Marie Le Riche de Prony (1755–1839), Antoine Chézy (1718–1798), Pierre-Simon Girard (1765–1836), Jean-Charles Borda (1733–1799), Pierre Louis Georges Du Buat (1734–1809), and others.

In Italy we remember Giovanni Poleni (1683–1781), Francesco Maria De Regi (1720–1794) and Father Barnabite Paolo Frisi (1728–1784). Giovanni Poleni —may be best remembered well for his studies about the reinforcement of St. Peter's Dome in Rome (Benvenuto 1991)— studied some problems related to water efflux from an orifice and then he stated the weir laws (Poleni 1717), De Regi focused on the measurement of running water (De Regi 1764), while the Barnabite Frisi devoted his life to study river hydraulics (Frisi, 1770; Frisi 1777).

Figure 7
Poleni: *De Motu Aquae Mixto* (1717)

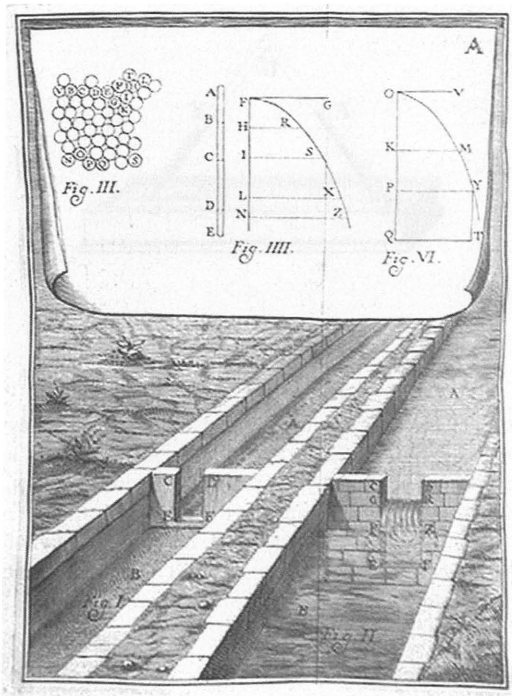

Figure 8
Belidor: *L'Architecture Hydraulique* (1737–39)

As the Italian School of experimental hydraulics we remember a great number of scientists and among them we mention Ottaviano Cametti (1711–1789) (Cametti 1777) and Niccolò Carletti (II half of 17th) (Carletti 1780). In this scientific context, it's very important to remember the researches of Jacopo Riccati (1676–1754) on the problem «to determine the force caused by fluid bodies crashing into solid bodies» (Riccati 1742) or, also, his studies «on the laws of fluid resistance to delay the motion of solid bodies» (Riccati 1722). The Guglielmini's works and his «Epistola hydrostatica» (Guglielmini 1731) written by Domenico Guglielmini (1655–1710) in 1731 have to be remembered. And we also remind Anton-Maria Lorgna (1735–1796) and his researches on running waters (Lorgna 1777), Antonio Rocchi (1724–1780) and his studies on measurement of bodies velocity and strength in motion and his application to hydrostatic problems (Rocchi 1775), Gregorio Fontana (1735–1805) and his dissertation upon hydrodynamics, on the motion of a body in a resistant medium, on the waterproof of channels (Fontana 1802), on the water pressure in motion into vessels, tubes and pipes, on the effect of centrifuge force on fluid motion (Fontana 1803), and the mature studies carried out by Louis Lagrange at the early eighties of 18th century (Lagrange 1781; Lagrange 1781–85). Finally, we have to mention the translation of Hydrodynamics treatise of abbé Charles Bossut (1730–1814) by Gregorio Fontana (Fontana 1785) in 1785.

In France, after the important studies begun by Mariotte and *le Chevalier* Samuel Morland (1625–1695) (Morland 1685), we remember the scientific work of Claude Antoine Couplet (1642–1722) on the resistance of pipes subject to great pressure, and then the very editorial effort of Bernard Forest de Belidor (1693–1761), who published an important encyclopaedic treatise on *Architecture hydraulique* (2 volumes in 4 tomes) where he thouroughly investigate on subjects related to hydraulic engineering and construction machinery (hydraulic wheel, watermill, windmill, suction pump, water pump, hydraulic pump, vessels, tubes, pipes, and others topics as maritime construction as weirs, dams, channels, river ports, and others).

The difficult field of mechanical science was a challange for many other scientists who obtained a large number of interesting results. Henri de Pitot (1695–1771) invented an instrument to measure running water; Antoine Chézy (1718–1798) expressed a mathematical formula for the evaluation of water velocity in a channel under constant running water. This formula is still in use in applied hydraulics. John Smeaton (1724–1792), a famous English engineer, was involved in experimental hydraulics; Charles Borda (1733–1799) carried out several laboratory tests on fluid resistance and on liquids' efflux from one or more orifices in a vessel. Charles Bossut (1730–1814) conducted extensive studies on hydrodynamics (Bossut 1775) and on experimental hydraulics (Bossut 1795); Pierre Louis Georges Du Buat (1734–1809) studied various phenomena related to fluid motion through pipes and channels and so he described velocity of water efflux from an orifice, pipes resistance under constant pressure and then he developed a semi-empirical method to evaluate channels' cross section in accordance with Chézy' theory. Finally, we mention the contributions of Giovanni Battista Venturi

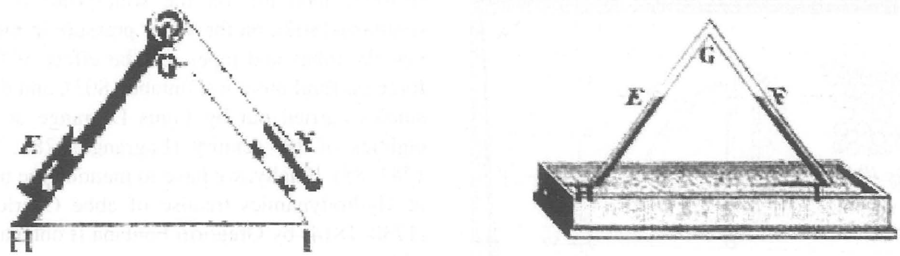

Figure 9
Le Chevalier Morland, *Elevation des eaux . . .* (1685)

(1746–1822) and Reinhard Woltman (1757–1837) with their studies on measurement of pipes resistance under pressure and their researches on water motion in tubes and channels (Rouse and Ince 1954).

4. New trends in hydraulics

By the end of the 18th century and the beginning of the 19th century, a large number of studies and researches on hydraulics (hydrostatics and hydrodynamics) was carried out. This trend was set with an imposing amount of treatise, books, critical essays on various topics related with hydraulics and its applications to engineering. The work of Pons-Joseph Bernard (1748–1816) on *Principes d'Hydraulique* (Bernard 1787), the treatises of Gaspard-Clair-François-Marie Le Riche, Baron de Prony (1755–1839) on *L'ecoulement des fluides incompressibiles* [Prony, 1790–96], on the *Jaugeage des eaux courantes* [Prony, 1802], or on the *Théorie des Eaux Courantes* (Prony 1804), the essay of Pierre-Simon Girard (1765–1836) on the *Mouvement des eaux courantes* (Girard 1804), or the critical review of Du Buat's treatise by François-Michel Lecreulx (1729–1812) (Lecreulx 1809) are an anticipation of the publishing revolution which enhanced the diffusion of the new trend in hydraulics scientific community. The Prony's *Nouvelle Architecture Hydraulique* (Prony 1790–96) so as the Belidor's *Architecture hydraulique* (Belidor 1737–53) were the basis of applied hydraulics in engineering (Belidor 1729). They had a great diffusion in scientific community and in practical engineering as well. They turned theories from Bernoulli, Bossut, d'Alembert, Euler, Lagrange into construction of machinery and instruments to convey water in rivers, channels, tubes, pipes, etc. It is a revival of applied hydrostatic and hydrodynamics, glorys for Italy and France in the Renaissance.

The following century will be the century of theoretical hydrodynamics or fluid mechanics, but this is another story.

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Forms and construction methods of some ancient dams in the Comunidad Valenciana

Rafael Cortés

The great importance that water has played in all cultures, especially in those countries with unfavourable climates, is undeniable.

The easternmost area of the Iberian Peninsula and within it large areas of the Comunidad Valenciana may be classified as sub-arid. These particular climatic conditions produce irregular rainfall which has greatly conditioned the form of the rivers which drain the land.

The majority of the most southerly drainage basins receive annual precipitation of less than 400 mm and extensive areas with an average less than 300 mm. Additionally, if we bear in mind that this rainfall is usually concentrated in short time periods, the result is that floods occur with some frequency, especially in autumn.

These circumstances, along with the fact that population was principally found in river and coastal areas during certain periods, led to attempts from early times to control the irregular river flows to guarantee the supply of water to populations as well as the supply of irrigation water during the long periods of drought.

Cultures with a great tradition of water management such as the Arabs and the Romans obviously left a profound legacy. However, the continuity and intensity of the use of this infrastructure along with the destructive power of flooding has generated a regular succession of breakages, modifications, repairs and reconstructions, which often make impossible the task of recognizing the original constructions.

Much is the case with numerous diversion weirs, many of which are still in use and located on the final stretches of rivers such as the Turia, the Júcar or the Mijares and which for the aforementioned reasons will not be dealt with in this article. The article will focus the question on a group of dams; Tibi, Elche and Relleu —constructed between the 16 and 17 centuries in the province of Alicante and which constitute some of the most emblematic examples of ancient dams (excepting those of the Romans) not only within the Comunidad Valenciana but all Spain.

Forms of the dams

These three dams, to which we may add that of Almansa (as much for its similarity of layout as being contemporaneous with that of Tibi) display many common aspects, while together signifying a milestone in the history of hydraulic engineering.

The most important common characteristics are: their curved layout, the system of galleries for the clearing of mud, the fabric used (based on masonry covered with dressed ashlars) and the fact of being emplaced on rivers of low average flow, heavy flooding and with wide areas of their watercourses made up of erodable detritus.

Nevertheless, the form and construction of each of these dams present some differences which should be pointed out.

Tibi dam

Work began on the Tibi dam in 1580 and merely one year later it suffered its first interruption due to economic difficulties which lasted nearly 10 years, a period which additionally served to modify the geometric characteristics following a bitter debate between the various experts who studied the construction. Although agreement was reached in 1588, the work recommenced in 1590 under the direction of Cristóbal Antonelli, one of the engineers of King Felipe II. The works reached their culmination around the middle of 1594 at a height of 196 palms when the town requested authorization from the King to conclude the works due to the lack of necessary resources required to reach the projected height (Alberola 1994).

The dam functioned normally until the great breakage of 1697 and there is only one report of a breach in April 1601 (Bendicho 1640), which, it seems, was due to the incorrect handling of the outlet sluice gate.

Many authors have dealt with the causes of this breach (Maltés and López 1752; Cavanilles 1795; Viravens 1876; Camarero, Beviá and Beviá 1989; Alberola 1994) and in the documentation supplied it seems that various circumstances came together. In the year 1688 the silting-up of the reservoir was so great that only 22 palms remained usable (less than 5 metres). Consequently, the decision was made to open a breach «through the cavity wall where the water governor was, being the thinnest part» (ACA. Consejo de Aragón Leg. 863–2/4) so that of the water would drag away part of the deposited sediment. The breach was closed with flagstones that were incapable of resisting the floods produced in 1697, which, combined with the weakening of the wall and the deterioration of the lime mortar caused by the circulation of water through it produced an important breakage and the emptying of the reservoir.

The dam remained unrepaired until the year 1736, when the works began which would finally prove definitive. Notwithstanding, in this extended period of time various reconstruction proposals were undertaken: Juan Blas Aparicio in 1698, Próspero de Verboom in 1721, José Terol, Vicente Mingot, Nicolás Puerto, Francisco Asensi and Juan Bautista Borja in 1733 and Pedro Moreau in 1733. In all of them, the breakage that the dam had suffered was represented along with proposals for its repair, all of which allows us to see the form of what has been preserved to the present-day as the most important changes have only affected the drainage systems and not the principal structure (Camarero et al. 1989).

The initial construction, interrupted at only a few metres height, possessed a rectilinear upstream face and a curved downstream face. This idea was rejected by all who made proposals for its recommencement, imposing instead a curved layout with an average radius of 90 metres. The upstream face has a gentle slope of 0,75/10, whilst the downstream slope shows six steps distributed in the following manner: a slope of height 18,6 m from the base, a first step of 1,0 m width and 5,25 m height, the second of 1,0 m by 4,5

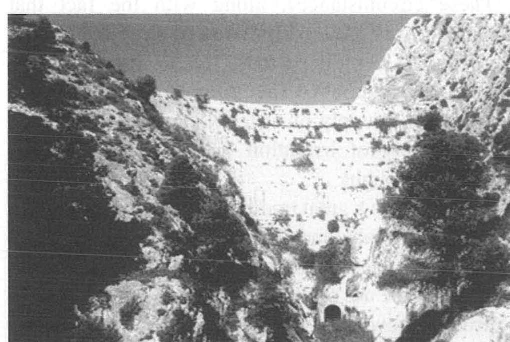

Figure 1a
Tibi dam according to Cavanilles. Figure 1b. Tibe dam in 2002 year

m, a third of 0.8 m by 3,3 m, the fourth of 0,6 m by 2,5 m, the fifth of 0,5 m by 2,6 m and the sixth of 0,4 m by 2,8 m to finish off with a bevel of 1,3 m height.

The total height is 42,7 m, the width being 33,7 m at the base and 20 m at the crest.

With respect to the drainage mechanisms, there is a surface weir with two openings situated on the extreme right of the crest, a water intake through an embedded well in the dam placed one metre from the upstream face with 52 pairs of slits and a base sluice gallery with dimensions of 1,8 m width by 2,8 m height at the intake widening rapidly in three metres to 3 × 3,3 m and progressively increasing its size until it reaches 4,0 m. width and 5,85 m. height at the outlet.

The theoretical capacity of the reservoir is 3,7 Hm^3 and, although at the beginning of the 20^{th} century an attempt was made to increase it, the scarcity of provision from the river Montnegre caused the idea to be abandoned. What was undertaken was the substitution of the clearing gallery for another excavated into the rock of the left edge, provided with two sliding panels. These works were begun in 1934 and concluded in 1943. Some years later, in 1945, a campaign of injections into the body of the dam was undertaken to reduce the filtration that had been detected.

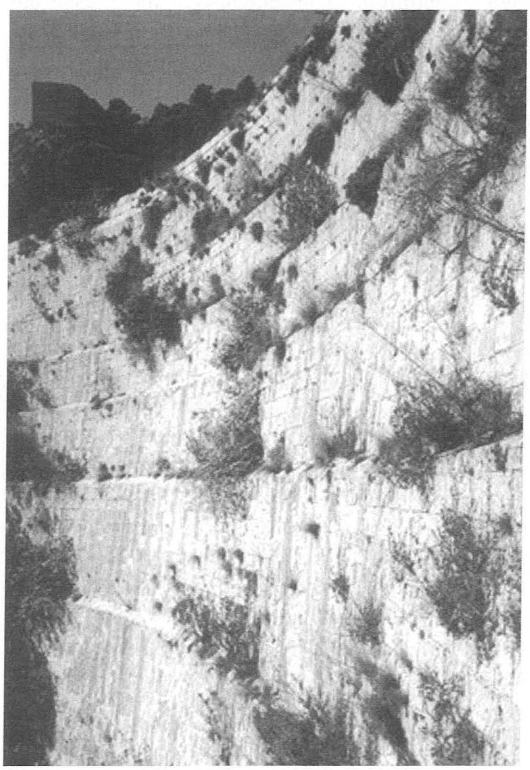

Figure 2
Tibi dam. View of the downstream face

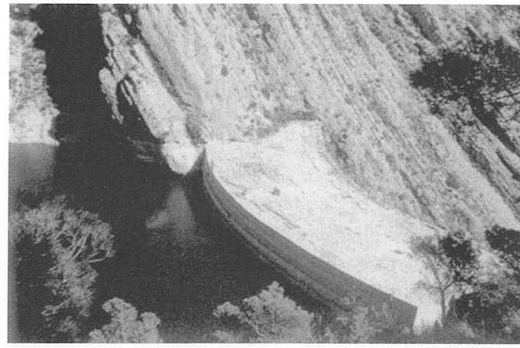

Figure 3
Tibi dam. General view of the crest and spillway

Elche dam

The first project dates from 1589, the year in which the council of the town of Elche, faced with the reduction of available irrigation water from the river Vinalopó (probably due to the intensification of the use of upstream springs) formed a committee of experts to study the watercourse and undertake the formation of a proposal for a reservoir. This was made by Joanes del Temple and Pere Izquierdo, who had participated in the projects of Almansa and Tibi respectively, along with the local masters Domingo Chavarría and Miguel Sánchez. Despite having obtained the necessary authorization from the ecclesiastical and civil authorities in 1590, the commencement of the works was delayed by the elevated cost of the Tibi dam, the rupture of 1601, and the decrease in the area of cultivated land as a result of the expulsion of the Moriscos (Gozálvez 1977).

The emplacement of the dam was proposed on the narrowing of the river at the point of a hill named

Figure 4
Elche dam. General view from the left edge.

Castellar de Morera where rock outcropped throughout the enclosure. The principal stages of construction correspond to the periods 1632–1640 and 1643–1655, although the reservoir was not filled until the year 1672. (Irles, Jaén and Irles 2002).

The operation of the reservoir had numerous problems principally derived from two factors: the silting already mentioned in 1653 and the salination of the waters. To this we must add the important floods which sporadically occur in the river Vinalopó, contributing to the increase in sediment loads and frequently provoking faults in the gate of the drainage gallery.

All this led to the drawing-up in 1670 and 1732 projects for the construction of a diversion canal so that river waters could avoid the reservoir although the diversion was not undertaken until the 20th-century.

The silting and floods resulted in the filling of the reservoir with mud, which around 1751, rendered it unusable. Various remedial projects followed: that of 1762 with the change in the system of the closure of the base outlet or that of 1793 to reconstruct the sluice gate and vertical sliding door. In the same year Cavanilles visited the lake, finding it with the sluice door open, empty and silted-up. (Cavanilles 1795). Throughout the 19th-century the periods of disuse under repairs alternated with periods of precarious operation until the Elche council opted for the transfer of the lake to the owners of Acequias Mayor y Marchena who refurbished the dam in the same year, without modifying the previous structure since the description of Cavanilles matches very well with the appearance that the dam presents today.

In accordance with the proposals of the civil engineer Prospero Lafarga (Lafarga 1910) between 1906 and 1910 a diversion of the river Vinalopó was undertaken before its entrance to the reservoir. Starting from the mill weir of Pavía a canal was constructed with a total length of approximately 4 km, of which 1.9 km were tunnelled and was in operation until the '40s when floods put the intake out of use.

New projects were subsequently drawn up to install a new base drain and to refurbish the intake of the diversion canal. However, only the latter work was carried out. The reservoir, little by little, lost interest for the irrigators as a consequence of the employment of other waters originating from the Tagus-Segura transfer, with the consequent abandonment of the exploitation, resulting in the spontaneous rupture of the base sluice drain due to its poor condition. Consequently, the reservoir proceeded to empty, carrying with it an estimated volume of 100.000 m^3 of mud (Zaragoza 2002).

At the present time it seems that various refurbishment projects are underway as much on the reservoir itself as some of its accessory works which, in any case, must be compatible with the declaration in 1999 of the Generalitat Valenciana of the dam of Elche as an Asset of Cultural Interest(Irles, Jaén and Irles 2002).

With reference to the dimensions of the dam, it presents some peculiarities of great interest. It consists of two clearly differentiated parts. The main body, formed in layout by two curved sections and another smaller body situated alongside the right edge which, it seems, functioned at some time as a weir with a fixed lip, although at some later time it was heightened to the crest level of the principal dam.

The main body of the dam is formed by an arch with a 63 m. radius with a height above the watercourse of 23.2 m, width at base of 12 m and 9 m on the crest the inclinations being 0,049 and 0,092 upstream and downstream respectively. Some authors (Fernández Ordóñez 1984) consider it to be the first arch buttress dam in the world since its stability is precarious unless one considers the effect of the arch.

On the left edge the arch rests its lower section on the rock of the bank whilst the upper metres are closed with a straight wall positioned upstream

according to the radial alignment. The other side of the arch rests on a rocky promontory existing in the center of the enclosure on which a buttress has been placed to improve the rim.

In addition, there is another arch of much reduced dimensions which encloses from a buttress to the bank of the right edge.

Around 100 m from the main body on the right edge there is a small hill enclosed by an arch wall 11m in height and 7 m width at the crest, although it seems that the water level was 1 m lower than that of the principal dam when it functioned as a Weir.

The base drain constitutes a Gallery of 2,2 m × 2,7 m with half point vault, in which there were two gates, one near to the upstream face and composed of a rectangular sluice gate secured with horizontal beams sunk into lateral holes in the ashlar courses. The other was a vertical sluice gate operated from the sluice chamber situated above the base gallery and secured with horizontal beams and a large vertical log.

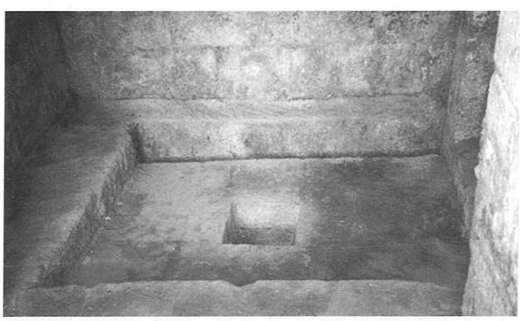

Figure 5
Elche dam. Interior of the operations chamber

The water intake consists of a well of 1 m in diameter, embedded into the body of the dam, with slits in alternate courses and height coincident with that of the ashlar and a width of 10 cm. This structure is not the original but corresponds to an important repair work carried out in 1764.

The theoretical capacity of the reservoir is 4 Hm^3 though by the mid —20th-century the thickness of the silt deposits reached around 18m resulting in an estimated capacity of around one tenth of the theoretical capacity.

Relleu dam

This work is far less studied the previous two and the author of the project and date of construction are unknown. It is known that in 1653 the town of Joiosa managed and obtained a privilege of King Felipe IV, authorizing the construction of a reservoir on the river Amadorio. However, various authors do not agree on the year in which it was constructed offering various dates: 1628,1629 and even the 18th-century without supplying any reference with respect to the origin of the information (López Gómez 1987). Sendra (1964) gathers together an application of 1777 made by the population of Relleu to interrupt the heightening works on the dam, which could be one of the causes of the confusion about dates.

The work is emplaced on the river Amadorio on an impressive stretch immediately after the confluence with the creek of Marulles. It is a dam of curved layout with a radius of around 60 m, vertical faces and thickness of 10 m. Initially, it had an approximate height of 28 m and was later heightened in 1879 to 31,85 m with a wall of four metres thickness.

The volume of the reservoir, after the heightening, was 600.000 m^3 and although, now in the 20th-century, new enlargement plans were drawn up for the reservoir none of them was executed and, in 1961, with the entrance in operation of the reservoir of Amadorio of 8 Hm^3 capacity situated a few kilometres downstream, the reservoir of Relleu ceased to be useful apart from the purpose of sediment retention and fell out of use.

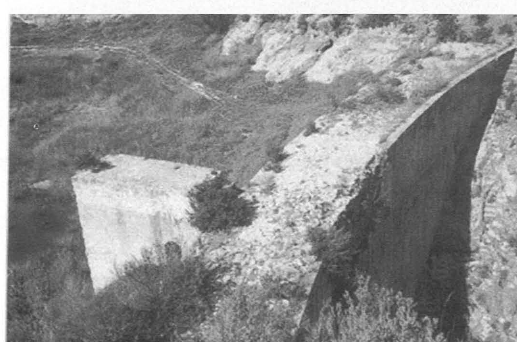

Figure 6
Relleu dam. View from the right edge.

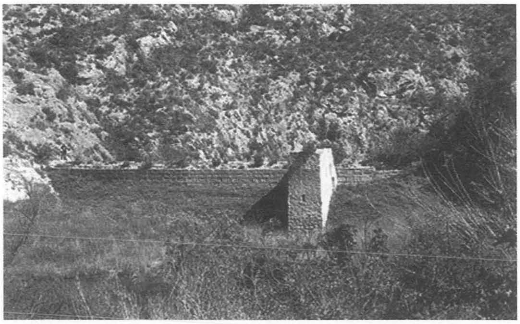

Figure 7
Silting-up of the reservoir of Relleu , seen from upstream

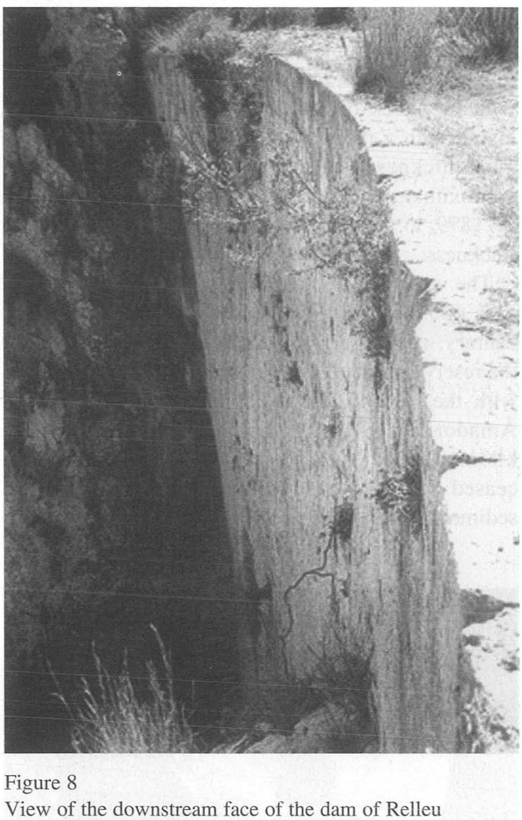

Figure 8
View of the downstream face of the dam of Relleu

Water intake was achieved through a well embedded in the body of the dam, which, on the heightening the dam with only four metres thickness, was constructed as an attached tower. It is equipped with slits of 0,25 m × 0,25 m , arranged regularly in the face to a height of approximately 9 m from the base. This is due to the fact that the enclosure is very narrow at the base of the reservoir and, until the water reaches to said water level, it doesn't begin to open out until reaching the almost 40 m width of the crest.

The well is connected to the clearing gallery, which consists of a tunnel of some 2 m width, with an opening in the upstream face of 1,0 m × 2,0 m closed with a wooden sluice gate. Similarly, it possesses a operations chamber connected to the sluice gate by means of a well to which access is gained by means of a sloping gallery whose entrance hole in situated in the downstream face very close to the left bank and to which access is gained through some steps cut into the rock of the said bank.

Construction aspects

The masonry of the three dams is very similar, the main core is composed of masonry bedded with lime mortar and rendered on both sides with a quality face of calcareous ashlar. Nonetheless, some small differences exist which, in many cases, correspond to the many repair works suffered.

The large quantity of existing documentation on the dam of Tibi allows us to know of some construction aspects which, doubtlessly, may be applied on general lines to other works. In the response of the experts to the memoir of 27 points of conflict over the work of Tibi, dated 7 Dec. 1587, it is recommended that « . . . In the front as in the lower part to carry on with segmental arches and that in the body of the work that the official, or officials should undertake the construction of masonry arches using appropriate seater stones on the rims» similarly, it was recommended to render with bitumen the joints in the faces in contact with water, from the center outwards «as is usually done in cisterns»

In the study which Próspero de Verboom conducted in July 1721, for the repair of the dam, it was recommended to use ashlar blocks of a greater size than was the custom in the locality as well as joining them with iron clamps set in lead in the most delicate areas. An interesting suggestion is made that it would be useful to bring puzolana from the kingdom of Napoli for use on the first face of the work in order to give it greater solidity as well as giving a more rapid entrance for water into the lake.

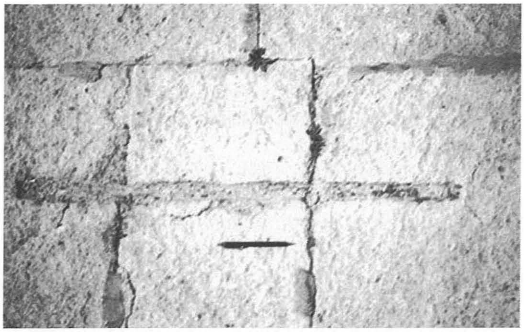

Figure 9
Elche dam. Detail of clamps on the crest

In the report for the refurbishment of the work, whose purpose was to give it its definitive form, undertaken in 1733 by a team of experts: José Terol, Vicente Mingot, Nicolás Puerto, Francisco Asensi y Juan Bautista Borja with the overview of the military engineer Pedro Moreau, some chapters or sheets of conditions were included and would prove very enlightening us to what was considered good construction in this era. These chapters comprise various sections where the following aspects are dealt with. lime, sand quality, mixing of lime, masonry, «broken stones», dressing of ashlar on general chapters to observe at work. Of all the conditions that the materials had to comply with and their use in the works, the most relevant refer to the placement of lime ovens on site; no material could be unloaded or placed in the works without previously being seen by the person responsible; with the lime three distinct types of mortar were made: for fixing, for masonry and for concrete; the mixing of the lime had to be done in the last quarter moon, one or two moons before its use, be of a greasy consistency and placed in the works in such a manner as to be able to be pressed with a tamper without the addition of water; the sand used had to be free from salt and stones; the masonry stone must not be extracted from areas which would prejudice the ashlar quarries, be of good quality and of varying sizes; the ashlar blocks would have to be extracted from the quarries indicated by the superintendent with the correct size and shape, the dressing had to be done by chisel and faces by pick, and perfectly squared on all faces; the hardest stones were reserved for conduits and sluice gates.

In the general chapters, the buyer was obliged to remove all areas that appeared to be in poor state, as well as to give special attention to the connections between old works and new works. To achieve this, it was necessary to unite the new courses with the old. Additionally, in the areas where forces were greater, it was necessary to unite them using iron clamps conveniently set in lead. In cases where it was necessary to set in the rock, they had to excavate the shape of the block to guarantee it's perfect collocation. Similarly, in the masonry it was necessary to seat the stones properly with the blow of a hammer and overseat with a tamper so as not to produce the later seats.

Finally, among other conditions referring to the water intake and the drainage gallery, it was necessary to clean vegetation and roots from the crest, lifting all the courses that were necessary until

Figure 10
View of the quarry on the right edge of the dam of Tibi

Figure 11
Quarry of the dam of Elche. Left edge

arriving at the clean area later replacing the blocks so that the work remains level.

The quality of the materials used in the three dams must have been very similar, as all have tolerated the passage of numerous floods over the crest without appreciable damage. The limestones used in Tibi as well as the myocene calcretes extracted from the quarries still visible on the left edge of the dam of Elche over the numulitic limestones used in Relleu are of sufficient quality to have maintained the aspect of these works and despite the passage of years, often in neglect and abandonment.

Unfortunately it is necessary to point out another characteristic common to these three works: the poor signposting and bad condition of their respective access roads, which, perhaps, shows an apparent lack of interest for hydraulic heritage despite some of the dams being declared Asset of Cultural Interest by the corresponding administration.

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The façades in the traditional rural constructions and their study through the documentary sources. Application to the house of principles of XX century in the region of Paradanta (Pontevedra, Spain)

B. Cortés Redín
J. Ortiz Sanz
I. Cañas Guerrero
T. Rego Sanmartín

The basic type of the buildings in the so-called vernacular architecture, typical of each area or sub-area, has to be established through a process of analysis and synthesis of the greatest possible number of buildings (García Grinda, 1990).

When dealing with the analysis of the contemporary traditional rural buildings, nowadays the necessity arises of looking for valid documentary sources to complete the compulsory field evaluation of the typological resources of the landscape (Rivera 1992).

This type of documentary sources is specially useful when you need to analyse a high number of buildings in areas of disperse construction, where the fields study would imply a high cost (Ortiz 1999).

Among the documentary sources we can highlight the cadaster inventories, owing to the great amount of information they provide. Of all the cadasters made until today in our country, we have used in this study the «Libretas de campo de aparejadores» (technical architects field notebooks) which were made in all the parishes of Galicia in 1920 and which were used for the subsequent tax register of buildings and lands.

The aim of this paper is to show the information these field notebooks provide and also the possibilities they offer when having to analyse the earlier XX century traditional rural buildings in our country.

Material

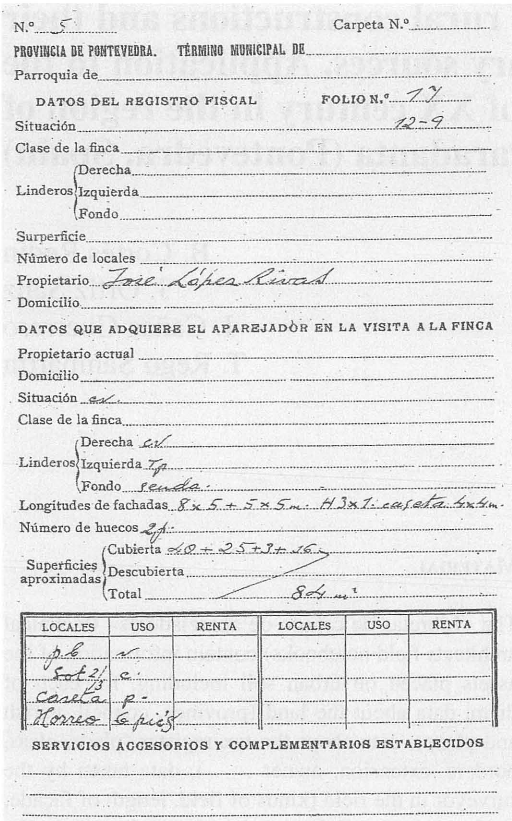
The «Libretas de campo de aparejadores» (technical architects field notebooks) contain information of the assets placed on urban soil including, for each of them, data about the land (province, council, parish and place), data about the tax register (place, class, borders, extension, owner . . .), data taken by the surveyor in the field (kinds of field, length of façade, roofed and unroofed extension . . .), data about the building materials and ways (wall materials, wall coatings, decoration roof, floors, paving . . .) and a drawing of the building.

These data are shown in an index card where the technicians would note down those aspects which were relevant (Figures 1 and 2). In Figure 3 some examples of drawings of buildings from the field note books have been included.

To carry out the study, we took as a sample the façades of all the villages in a parish in the area of «A Paradanta» (Pontevedra, Spain), which in the end implied the study of more than 500 façades.

Method

To carry out the analysis we had to define first the different buildings and façades that could be found in the register. With the help of a data base, we finished off the analysis of all the buildings and façades in the sample.



N.º 5 Carpeta N.º

PROVINCIA DE PONTEVEDRA. TÉRMINO MUNICIPAL DE

Parroquia de

DATOS DEL REGISTRO FISCAL FOLIO N.º 17

Situación 12-9

Clase de la finca

Linderos: Derecha / Izquierda / Fondo

Surperficie

Número de locales

Propietario José López Rivas

Domicilio

DATOS QUE ADQUIERE EL APAREJADOR EN LA VISITA A LA FINCA

Propietario actual

Domicilio

Situación

Clase de la finca

Linderos: Derecha / Izquierda / Fondo

Longitudes de fachadas 8 x 5 + 5 x 5 m. H 3 x 1: caseta 4 x 4 m.

Número de huecos

Superficies aproximadas: Cubierta 40 + 25 + 3 + 16 / Descubierta / Total 84 m²

LOCALES	USO	RENTA	LOCALES	USO	RENTA

SERVICIOS ACCESORIOS Y COMPLEMENTARIOS ESTABLECIDOS

Figure 1
Example of an index card to take data in the field - I

Descripción de la finca

Materiales y sistema de construcción:

Muros / Revoco exterior / Decoración id. / Cubiertas / Pisos / Pavimentos / Escaleras

Fachadas / Tabiques / Revestido interior / Decoración id. / Carpintería de taller / Armaduras / Cielo-raso / Cocinas

Observaciones

Día 10 de Nobre de 1923

El Aparejador,

CROQUIS

Figure 2
Example of an index card to take data in the field - II

The data base is composed of four groups of fields.

The first group of fields is devoted to the identification of the studied sample. Each of them would then be identified by the municipality, the parish, the place, the index card relevant in the place and the identification code of the card.

As more then one building can be registered in a card, it was necessary then to include another group of fields which enabled us to consider this aspect. In this way, a register could be created for each building being analysed. Besides, it was also noted down whether the building was attached to another one or not.

The third group of data was devoted to the typological study of the building. To carry it out we considered the criteria of Ortiz et al (2001) about the building kind of development, plan shape and number of roof slopes.

The building development is produced from a main volume in different ways (Figure 4). The first one is called simple development and in it the new volume is attached to the gable end wall of the former, following the longitudinal axis of the main volume. The second one has been called complex development and in it the new volume is attached to one of the longitudinal walls of the main one. In the second kind of development we can find three situations:

— Complex development through perpendicular attachment: the longitudinal axis of the new

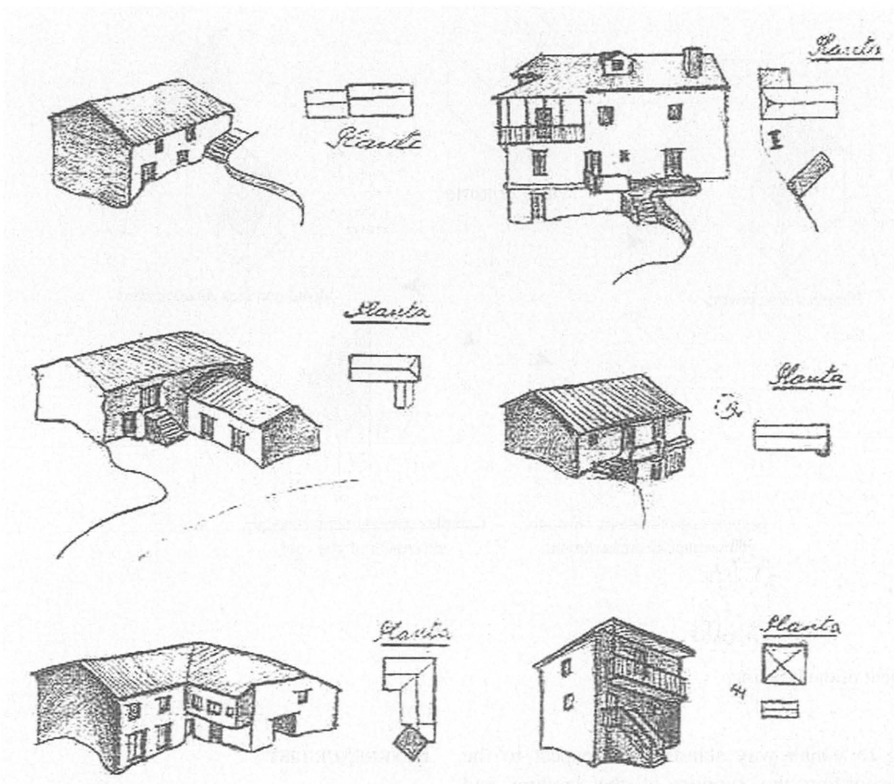


Figure 3
Example of several building drawings

volume is perpendicular to the one of the main volume.
— Complex development through extension of the roof: the longitudinal axis of the new volume is parallel to the one or the main volume.
— Mixed complex development: one of the end of the main volume is developed through extension of the roof and the other one through perpendicular attachment.

As far as the shape of the plan is concerned, we find plans with an «L», «U», «T», etc, shape. As far as the roof slopes are concerned, these are noted down in each wings. The wall materials, the roof materials and the build surveys were also included for their analysis in the data base.

The fourth group of fields in the data base would be constituted by the data of the analysis of the façade itself. In the first field the façade is given a code because one or two of them can be analysed in each building according to the characteristics of the drawing. Once each of the façades had been identified, we analysed those aspects which have a higher relevance in the typological definition of the composition of the façade. The two following analysed aspects are the shape of the baseboard and of the eave. These can be flat or sloping. Then, it was studied which way the façades stands with respect to the roof. This can be perpendicular or parallel. The analysis of the height and length finishes off the study of the general characteristics of the façade. Next the total number and type of doors and windows is analysed. Finally, the remarkable elements in the façade, such as stairs and projections are noted down. The stairs is characterised

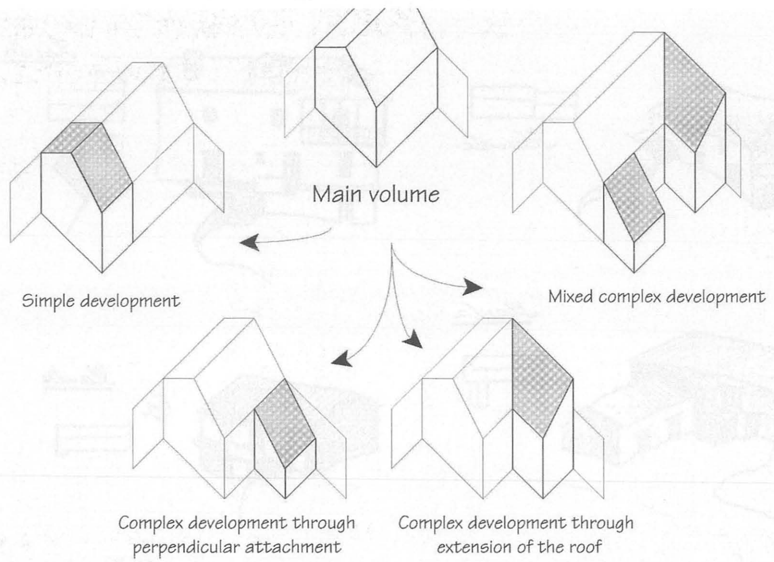


Figure 4
Development of the buildings

according to which way stands with respect to the façade, according the support of the landing and according its faces. The projections in the façade are characterised according to the way they are supported and according their faces.

Results and conclusions

The synthesis of the data obtained after the analysis allowed us to know the most relevant characteristics of the sample.

The results obtained were the frequency of the aspects studied. These distributions give us a very clear idea of which were the most frequent and then the most representative aspects of the buildings and the façade in the sample studied. These distributions would become the base for the establishment of the basic type of traditional rural buildings.

With this study it can be highlighted the great usefulness of the «Libretas de campo de aparejadores» (technical architects field notebooks) in the study of some remarkable aspects of the façades in the early XX century traditional rural buildings in Galicia.

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Gothic vaults: A rationalist or a tectonic track for the role of the ribs?

Anne Coste

TO VISIT AN ANCIENT DEBATE AGAIN

The construction history is also the history of the great constructive debates. The world of architecture knew very famous of them: from the Milano Duomo to the quarrel about Sainte Genevieve church in Paris, going through the project of Firenze Duomo cupola. In these occasions, to observe the confrontation of the doctrines and the knowledges is very interesting.

In France, a continuing historical constructive argument is especially adapted as concernings the thematic of this meeting: the question of the debated role of the arches, particularly the intersecting ribs, in the mechanical behaviour of a Gothic vault. This question is not new but it seems interesting to me to mention it at the occasion of the First international Congress of Construction History for at least three reasons. Firstly, it is a typical example of a constructive question: in the meaning of building (*mise en œuvre*) and in the meaning of calculation, it will allow us to touch on the development of the sciences of construction and their supply for architecture: each time new scientific models appears (like graphic analysis, photoelastic models, FEM calculation, . . .), the question of the ribs is asked again. Secondly, it is an exemplary debate with a mixing of doctrine and objective science and it illustrates the influence of the theories of the second half of the 19th century on the development of Modern architecture. Lastly this question has been studied in plural disciplines: history, architecture, engineering and even semiology, that makes this argument particularly interesting from the constructive history point of view.

The problem is not to know whether the system of ribs hold up the ceiling of the vaults as Viollet-le-Duc believed it. I don't want me to decide between the «combattants» and establish who is wrong and who is right, even if we have studied the case of Auxerre cathedral with a Finite Element Method approach, pointed out the fact that the ribs didn't held up the vaults but created troubles into the piers of the ceiling (tension area at the level of the haunches for instance). We purpose instead looking at the development of this controversy: how did it begin, how did it become impassioned, who took part in it, and how did it evolve with the progress of constructive science and new scientific models; how finally, a theory which has been built in another academic discipline came and gave help to a new «constructive truth».

Henri Focillon already showed the real historical impact of the question:[1] «(le problème) de l'ogive est d'une particulière importance parce qu'il intéresse les méthodes de la recherche et, par ces méthodes mêmes, la conduite et les disciplines de l'esprit. Il ne s'agit pas seulement pour nous, malgré l'intérêt fondamental de la critique instituée sur ce point, de savoir s'il faut abandonner ou sauver tout ou partie de l'admirable édifice archéologique de Viollet-le-Duc, mais de mesurer avec justesse les rapports de nos coordonnées, technique, plastique, vie des formes dans le temps» (Focillon 1945, 110).

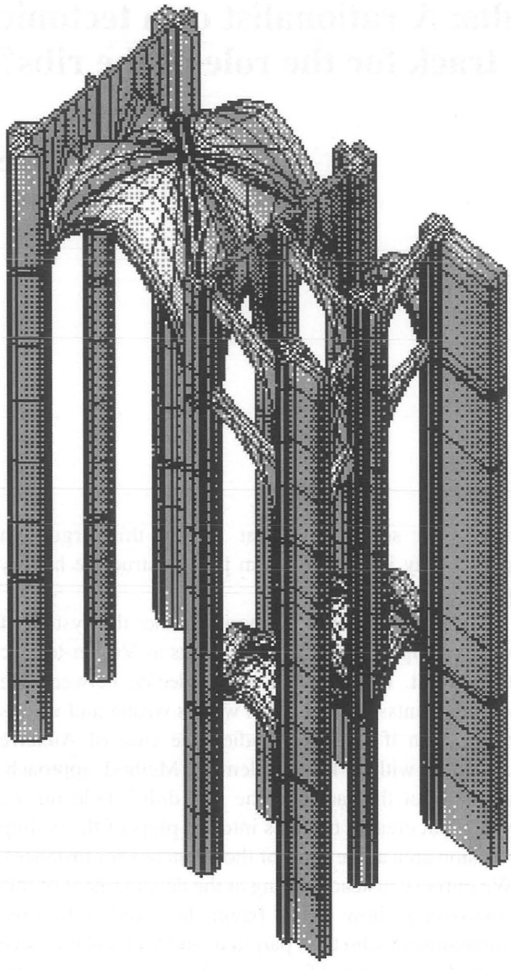


Figure 1
Modelisation for a Finite Element Method calculation (document : Anne Coste)

Nowadays we can read once again this debate clearing it up with Kenneth Frampton theory about the tectonic or the «poetics of construction» (Frampton 1996): if the Viollet-le-Duc's theory can be ranked in the side of a pure constructive determinism, the concept of tectonic allows us to establish, with Frampton, a continuity between form and construction and to give a real place to the relationship between constructive choices and spatial quality.

THE ORIGIN OF THE CONTROVERSY

The Viollet-le-Duc's rationalist theory described the ribs as a member canalising the load of the vault on the four angles in order to «transmit» it to the piers and to the pier buttresses through the flying buttresses. The main discussed point of this theory is the famous definition of the ribbed vault: « . . . suite de panneaux à surfaces courbes, libres, reposant sur des arcs flexibles . . . » (Viollet-le-Duc 1854–1868, vol. 4: 21). This definition poses two problems: the reality of the fact that the ribs held up the portions of the vault and their supposed «elasticity». I will go back over the tools used by Viollet-le-Duc in order to explain the behaviour of the vaults.

The first contradictors

The best known contributions to this argument are the thesis about «Viollet-le-Duc et le rationalisme médiéval» (edited in 1933) by the French architect Pol Abraham, and the engineer Victor Sabouret's article who asserted approximately the same thing a few years ago. In fact, the polemic started from 1900 with Brutails'book: he already wondered about the role of the intersecting ribs, bearing arch or cover-fillet?[2] He described the ribbed vaults of Morienval as a solution to a problem of masonry bond rather than a structural equilibrium problem. Somewhere, he noted that the diagonal ribs were thinner than the main arches of the vault and he found that the section of the nave's diagonal arches was identical to the one of the ambulatory in spite of the difference of their chord.

After that, A.Vaillant casted doubt on Viollet-le-Duc's theory: « . . . (il) n'a d'entendement qu'au service de son œil, que sa passion moyenâgeuse a déformé . . . » (Vaillant 1919, 89). Vaillant asserted that the Greek temple was the archetype of the whole of architecture, included Gothic cathedrals. This assertion is open to criticism as Brutails answered (Brutails 1920, 15), but he also introduced the doubt about the role of the intersecting ribs, as later Abraham theory: the first, Vaillant told that the ribs played only a role of masonry formwork during the mortar prism. It was Julien Guadet advice too (Guadet 1901, 323).

What did Viollet-le-Duc write about medieval methods ? «Il est probable que les architectes

gothiques primitifs s'étaient fait des règles très simples pour les cas ordinaires; mais il est certain qu'ils s'en rapportaient à leur seul jugement toutes les fois qu'ils avaient quelque difficulté nouvelle à résoudre. Comme s'ils eussent défini les lois des pressions des arcs, ils s'arrangèrent pour concentrer sur le parcours de ces lignes de pression les matériaux résistants, et, conduisant ainsi les poussées du sommet des voûtes sur le sol, ils arrivèrent successivement à considérer tout ce qui était en dehors comme inutile et à le supprimer» (Viollet-le-Duc 1854–1868, 64).

Viollet-le-Duc also thought that the method of dimensioning for the piers of a vault described by Derand in the 17th century was already known by the medieval builders. Their knowledge about structure and mechanics is still mysterious: the architects didn't know the notion of vector as a representation of a strength but they had a clear physical perception of strength, and they also knew about the qualities of materials before the science of Strength of Material was invented several centuries later.

Numerous authors have studied this question, with different trainings and functions: engineers, architects, historians of Art, all of them modified the knowledge of the gothic structural system during the 20th century. Because of the limited place in this paper, I will only skim over each contribution: for a deeper analysis, see Coste (1997).

I have to begin with Paul Planat's work. In the *Encyclopédie de l'Architecture*, he gave a personal interpretation of a gothic vault but the historians were not interested in this study. E. Rümler quoted it in his introduction of Pol Abraham's book: «Toutefois, il y aura demain cinquante ans, un savant et un vulgarisateur, P. Planat, cherchait à voir clair dans la structure compliquée de nos cathédrales, et le Dictionnaire d'Architecture de Viollet-le-Duc en main, il comparait les réalités que la science lui révélait, avec les théories trop souvent fantaisistes de l'auteur».[3]

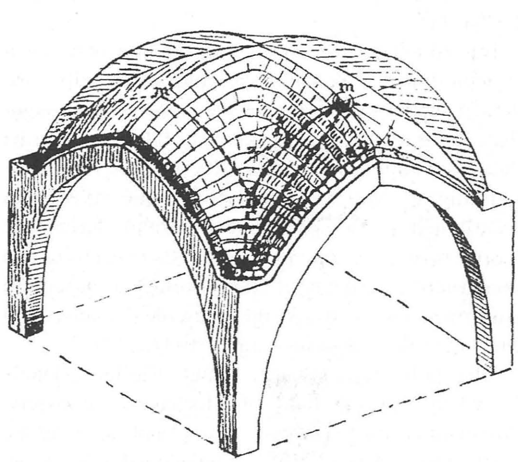
Pol Abraham himself, in the first part of his thesis (Abraham 1934a), cited the different archeologists and historians which have kept their distance from the Viollet-le-Duc's thesis. Firstly, he quoted the Anthyme Saint-Paul, J. A. Brutails and Marcel Aubert's work. All these studies are critical but very respectful reviews of the great architect Viollet-le-Duc's work.

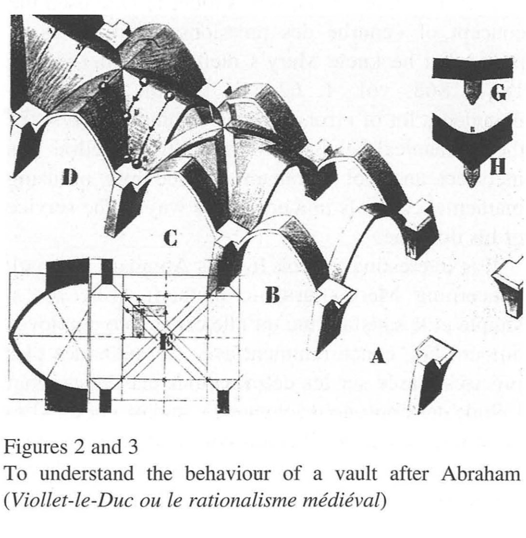
Abraham also quoted the work of engineers Paul Planat and Victor Sabouret, and René Schneider too (Schneider 1928).

POL ABRAHAM: «L'ÉQUILIBRE PLASTIQUE N'EST PAS L'ÉQUILIBRE STATIQUE»

Sabouret and Abraham's studies

Victor Sabouret published an article entitled «Les voûtes d'arêtes nervurées. Rôle simplement décoratif des nervures» in *Le Génie Civil* of March 1928. As





Figures 2 and 3
To understand the behaviour of a vault after Abraham (*Viollet-le-Duc ou le rationalisme médiéval*)

far as he's concerned, Pol Abraham had brought in the subject for his thesis from 1923 (Very 1989).

In the book *Viollet-le-Duc et le rationalisme médiéval*, Pol Abraham reproached Viollet for cheating over gothic architecture. After him, Gothic architecture was the contrary of rationalist: it was only an illusion, the diagonal ribs, the double walls and the colonnettes were an artificial system which concealed the real massive structure. «Aucune (architecture) n'a porté plus loin . . . l'illusion d'une structure idéalisée, à la base d'éléments linéaires adventices dépourvus de toute utilité matérielle. L'architecture gothique est donc, essentiellement, *une plastique*, et Viollet-le-Duc n'a soutenu, en conséquence, qu'un brillant paradoxe» (P. Abraham 1934a, 115).

But Abraham also made a distinction between a structural and a constructive role of the ribs. He denied the structural aim of the ribs but he thought that the constructive role was real: the diagonal ribs didn't hold up the ceiling but they were useful for the building of the vault and they may have also contributed to the general equilibrium during the mortar prism. The rare medieval texts concerning the constructive aspect of the Gothic architecture confirmed this analysis: they described intersecting ribs «to built vaults on» (Aubert 1934, 213).

The polemic was also about Viollet-le-Duc's knowledge in the field of Mechanics. Strangely, Abraham accused Viollet-le-Duc of not knowing the method that Mery (1840) had developed a few years ago (P Abraham 1934a, 10). Viollet-le-Duc used the concept of «courbe des pressions», that seems to prove that he knew Mery's method (Viollet-le-Duc 1864–1868, vol. 4: 62–64), but his calculation included a lot of errors. So, his technical analysis of the mechanical behaviour of a Gothic cathedral was incorrect and Pol Abraham accused him of using mathematical tools in a erroneous way in the service of his doctrine.

It is interesting to look for this Abraham's remark concerning Mery's graphic method: «Elle est si simple et si satisfaisante qu'elle est encore employée aujourd'hui, concurremment avec les méthodes plus précises, basée sur les déformations élastiques, pour l'étude des ponts en maçonnerie», and in a note: «Les expériences de la Commission autrichienne pour l'essai comparatif des voûtes en matériaux divers, faites en 1891, ont démontré en effet que les voûtes en maçonnerie se comportaient sensiblement, *sous certaines conditions*, comme des arcs élastiques» (Abraham 1934a, 10, note 6): it reminds us the notion of «flexible arches» that Viollet-le-Duc gave in the *Dictionary*. But Abraham himself pointed the mistake in «elasticity» and «plasticity» made by Viollet-le-Duc (Abraham 1934b, 254). Nevertheless, Abraham wasn't either a specialist of Mechanics and his own inconsistencies will be noted later by other authors (Mark 1982, 13).

Marcel Aubert's great contribution

«Il m'a semblé que le moment était venu de reprendre le sujet dans son ensemble et de rechercher ce qui, dans les théories anciennes, peut être modifié et ce qui doit être conserve»: in 1934, Marcel Aubert published an answer to Abraham and Sabouret's thesis in the *Bulletin Monumental* retracing the history of the ribbed vaults: he explained that some roman vaults built during the 2nd and 3rd centuries were equipped with reinforcements which looked like the future gothic ribs. Nevertheless, Marcel Aubert opted for the «canalising» role of the ribs: «(Les architectes Lombards) n'ont pas connu la véritable voûte sur croisée d'ogives dont les arcs, ogives, doubleaux, formerets, composés de claveaux et indépendants de la voûte qu'ils renforcent et dont ils ont facilité la construction, ont leur clef sensiblement sur le même plan, ce qui ramène aux quatre points de retombée les pressions que l'on pourra facilement épauler par des contreforts, des murs-boutants et des arcs-boutants, et permet, par conséquent, d'ouvrir de vastes intervalles entre les supports» (Aubert 1934, 12–13).

The author added a precious note in order to sum up the situation of the articles about this matter. In addition to Brutails' work (*Précis d'archéologie du moyen-âge*, 2e édition, 1924, pp. 133–134, et *Pour comprendre les monuments de la France*, 1922, p. 42), he quoted Clarence Ward (*Maedieval church vaulting*, Princeton, 1915): «il ne considère guère les voûtes que du point de vue de leur aspect extérieur et attribue leur origine et leur évolution surtout à des raisons d'ordre décoratif», A.K. Porter (*The Construction of Lombard and Gothic Vaults*); M. Roger Gilman, which had studied the Reims and Soissons gothic churches after the air-raids, he had noted the amazing strength of certain vaults after the collapse of the intersecting ribs and he conclued that

these members were only an aesthetic system («The Theory of Gothic Architecture and the Effect of Shellfire at Reims and Soissons», in *American Journal of Archeology*, t.XXIV, 1920)» (Aubert 1934, 205 note 2). Marcel Aubert also related the German archeologs Gall et K.H. Clasen's work (*Baukunst des Mittelalters, die gotische Baukunst*, 1932), «ne voyant trop souvent dans l'architecture que des combinaisons de volumes et de proportions, en dehors de toute nécessité de construction, (ils) supposent que les maîtres d'œuvre du moyen-âge ont seulement voulu prolonger sous les voûtes l'élancement des lignes verticales et se sont aperçu ensuite des avantages constructifs de la voûte d'ogives» (Aubert 1934, 206 end of the note 2). M. W. Van der Pluym (*Oudheidkundig Jaarboek*, 1932) denounced the untrue rationalism of the medieval architecture too.

At the end, Marcel Aubert didn't settle, he concluded his study about the role of the rib as following:

«Elle facilite d'abord le montage de la voûte et donne une sécurité certaine au constructeur pendant le tassement des mortiers. Elle renforce ensuite la voûte sur ses points faibles, le long des arêtes et sur le plan des sommets, et cela d'autant plus que les compartiments sont construits légèrement ou en matériaux médiocres». (Aubert 1934, 234).

A controversy which mobilized numerous authors

One year later, R. Doré resumed this controversy once again in the *Gazette des Beaux Arts*: «M. Abraham nous rappelle que les dégâts causés par la guerre ont montré qu'une voûte d'ogives ne tombait pas nécessairement lorsque ses nervures étaient rompues . . . Mais les surplombs considérables qu'on put constater dans les voûtes crevées semblent bien étayer, c'est le cas de le dire, la théorie de la voûte couvercle, étant entendu que l'homogénéité de la voûte a pu exiger de très longs délais. Ne voit-on pas même assez souvent les murs gouttereaux se déverser sans que la voûte d'ogives se lézarde, la fissure s'ouvrant le long des formerets ?» (Doré 1935, 122). Doré's analysis of the gothic vault is a little different: «Le génie des maîtres d'œuvre fut de tirer, après coup, d'une «commodité de chantier», un nouveau style d'architecture pour lequel nous professons tous une commune admiration» (Doré 1935, 125).

Pol Abraham and Victor Sabouret's theories were critized as soon as 1935 by another Ingénieur en chef des Ponts et Chaussées, Henri Masson. He disapproved on the hypothesis of the absence of tensile strength of the masonry and he thought that Abraham's method of calculation was out-moded (Masson 1935).

In 1939, the review entitled *Recherche* gathered new contributions. Henri Focillon, Pol Abraham, Walter H. Godfrey, Elie Lambert, Gurgis Baltrusaïtis et Marcel Aubert examined the problem of the ribs once again (Very 1989, 25).

After that, we find again this question at Princeton University with the famous study of Robert Mark, in about 1970. Mark used photoelastic models in order to solve the historical polemic about the role of the ribs and also the role of flying buttresses and pinnacles. He sided with Pol Abraham and asserted in conclusion that ribs are more decorative than structural (Mark 1982).

Then, Roland Bechmann purposed a complementary hypothesis (Bechmann 1981). The Suger's description of the building of Saint-Denis allowed him to develop an economist explanation for the role of the ribs: he thought that the invention of the rib was in fact the substitution of a stoned arch to the wood centring due to the lack of trees at this moment in the place where the ribbed vaults saw the light.[4]

THE OTHER WAYS

We have just seen the arbitration of the argument about the ribs from the Mechanics' point of view, by several authors (our presentation is not exhaustive). The problem was to understand whether ribs held up the ceiling, or whether they were decorative members. Now, we are going to explore new ways with authors who look for a more philosophic and maybe more architectural explanation of the use of the ribs.

A philosophic approach

After Erwin Panofsky, we can find the materialisation of the *Summa Theologiae* by Thomas d'Aquin into a gothic cathedral. In his book *Architecture gothique et pensée scolastique*, (1951) he compared the gothic space with the regulator principles of the philosophy

of the Middle Age. Panofsky alluded to the controversy: «S'agissant de l'architecture des XII^e et XIII^e siècles, l'alternative «tout est fonction, tout est illusion» est aussi peu pertinente que l'alternative «tout est recherche de la vérité, tout est gymnastique intellectuelle et oratoire» s'agissant de la philosophie de la même époque. Les ogives qui ne sont pas encore *singulariter voluti* ont commencé par dire quelque chose avant d'être capable de le faire. Les volées des arcs-boutants de Caen et de Durham, encore dissimulées sous le toit des collatéraux (frontispice), ont commencé par faire quelque chose avant d'être autorisées à le dire» (Panofsky [1951] 1970, 111). Panofsky developped the concept of «visual logic» about the gothic architecture.

A semiologic approach

Umberto Eco gave us a new key in order to understand this viewpoint: the concept of prime function and second function applied to the the ribbed vaults. He cited the debatted problem of the role of the rib that he perfectly knew, introducing the three hypothesis we have already seen: the rib holds up the ceiling, the rib doesn't hold up anything, the rib holds up the ceiling during the building as a temporary formwork. «Quelle que soit l'interprétation admise, personne n'a jamais mis en doute que la croisée d'ogives dénotait une fonction de soutien réduite au seul jeu des poussées et contre-poussées entre éléments nerveux et subtiles. La polémique concerne plutôt le référent de cette dénotation; la fonction dénotée existe-t-elle ? Si elle n'existe pas, la valeur communicative de la croisée d'ogives reste toutefois certaine et d'autant plus intentionnelle, voulue et valable qu'elle aurait été articulée seulement pour communiquer une fonction, non pour la permettre . . .» (Eco 1972, 276–277).

This very important remark reveals the real architectural quality of the gothic cathedrals. The rationalist theory of Viollet-le-Duc, as the most efficient numerical tools, are unable to give us an idea of the conceptual richness of that architecture. This is the new way that Kenneth Frampton follows in his recent book about the «Poetics of Construction». He also refers to Eco, quoting his theory of denoted function: « . . . One may also add that building, unlike fine art, is as much an everyday experience as it is a representation and that the built is a thing rather than a sign, even if, as Umberto Eco once remarked, as soon as one has an object of «use» one necessarily has a sign that is indicative of this use.» (Frampton 1996, 2). But the problem of the ribbed vaults in the Gothic architecture is more complicated because the denoted function of the rib doesn't fit a real structural function. Nevertheless, Frampton will allow us to introduce the last part of this paper with the importance of spatiality in Gothic architecture and the influence of Viollet-le-Duc's doctrine on the Modern Architecture.

Beyond the argument

The debate concerning the Viollet-le-Duc's rationalist doctrine is not closed, as Jean-Michel Leniaud's book shows it (Leniaud 1994), but we are going to explore another way in order to understand all the symbolic importance of the ribbed vaults system. Umberto Eco gave us an important contribution with the notion of denoted function but we can continue this reflexion about the ribs, wondering about their influence on the perception of the space.

The tectonic nature of Gothic architecture

Frampton noted that the notion of space, which is very significant in the theory of Modern architecture, was absent from the Viollet-le-Duc's pieces of writing. Yet spaciality is fundamental in the architecture of the great Gothic cathedrals. And I think that the system piers / arches, with little colonnettes starting from the soil and going to the ceiling with the ribs, is a very important member of this spatial quality. In order to define the role of the ribs, I wouln't speak about «structural» members but «structuring» members. These arches structure the space of the cathedral and, in the same time, they give us an illusion of a line of load, as Abraham said.

Viollet-le-Duc described the gothic architecture as a kind of ideal and operational model, projecting the leanings of his own time on the Middle Age and the possibilities of new materials on Gothic cathedrals in order to produce a doctrine which will be fruitful for the future (Bekaert 1980). Indeed, Viollet-le-Duc was a precursor of Modern architecture (Revel 1964): his work had hugely influenced the next generation of

architects, i.e. the generation of the Modern Movement. Pol Abraham himself detected this influence: «Viollet-le-Duc qui n'eut pas d'originalité plastique et dont l'imagination artistique était, en dehors du pastiche, d'une attristante pauvreté, trop intelligent pour ne pas sentir ses faiblesses, trouva sa voie dans une pure construction de l'esprit, paradoxale mais prophétique, celle d'une architecture qui, soumises aux plus étroites nécessités matérielles, en ferait, cependant, la substance même de la beauté. Admirable idée, puissante et féconde, et dont l'architecture internationale du XX^{e} siècle est, en grande partie, sortie. Mais, aussi, attitude de combat, machine de guerre contre l'Académie, dans une lutte où il fera figure de héros de l'art national et moderne contre la tradition gréco-romaine moribonde» (P. Abraham 1934a, 102).

This Viollet-le-Duc's influence on international modern architecture is very significant. Le Corbusier was one of the heirs of Viollet-le-Duc's theory. With the title of his book «Quand les cathédrales étaient blanches», he symbolically refered to the Middle Age as a time of revival but he didn't admit the gothic architecture as an ideal model. We know that Le Corbusier didn't like Gothic architecture much. And we find Pol Abraham once again on this subject with his sharp critic of *Vers une architecture*: «Enfin, M. Le Corbusier paraît tenir essentiellement à opposer l'art de bâtir, qui est fait "pour tenir" à l'architecture qui est faite «pour émouvoir». . . . Ce contraste n'est-il pas surtout théorique et n'est-ce pas, au contraire, de ce trop rare équilibre entre l'intelligence constructive et le sentiment plastique que sont nées les belles œuvres?» (Abraham 1924). We can notice that Pol Abraham was already sensible to the «Poetics of construction», i.e. the expressive potential of construction which seems to be the alternative way to the rationalist stand.

Let us Bruno Zevi bring the conclusion of this paper: «Ho rivisitato alcune cattedrali inglesi dopo che le bombe ne avevano infranto le vetrate e fatto cadere i riempimenti tra le costole delle volte: ebbene, quelle strutture, svincolate ormai perfino dalla trasparente cartilagine che le univa, sembravano aver realizzato a pieno il sogno degli architetti gotici: creare lo spazio, scandirlo, elevarlo, dargli forma senza interromperne la continuita» (Zevi [1959] 2000, 71).

Figure 4
Ribs of a vault after the collapse of the compartments (document : Anne Coste)

Notes

1. Focillon: the chapter VI «Le problème de l'ogive» was published a first time in *Bulletin de l'Office des Instituts d'Archéologie et d'Histoire de l'Art* in 1935 and, later, in a collective book, collection «Recherches», in 1939.
2. The author quoted Anthyme Saint-Paul: «L'arc-boutant est un peu à l'extérieur de l'église ce que la nervure est à l'intérieur; celle-ci est un cintrage permanent, celui-là un étai permanent» (Saint-Paul A. *La Transition*: 7).
3. Presentation by E. Rümler of Pol Abraham's book «Une thèse sur Viollet-le-Duc et le rationalisme médiéval». *La Construction Moderne*, 14 janvier 1934: 253.
4. This explanation was also used from 1911 by Arthur Kingsley Porter in his book *The construction of Lombard and gothic vaults*, New Haven: Yale University Press.

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Innovations in the design of fong-span building structures

Henry J. Cowan

Before iron became available as an economical structural material, the choice was between timber and bamboo, which had both tensile and compressive strength, but were easily destroyed by fire, and the masonry materials (stone, concrete, hard-burned brick and mud-brick or pisé de terre), which were fire resistant, but lacked tensile strength. These materials could be used over a significant horizontal span only in the form of an arch which was purely in compression.

The arch concept was known to the Ancient Egyptians, but they used it only for mudbricks. A catenary-shaped storage shed survives in the mortuary temple for Ramses II; its construction was abandoned when the Pharaoh died, so that this temporary arched structure, which would have been demolished in the ordinary way, survives.

However, this is not an isolated example. One sees to this day many mudbrick houses along the Nile outside the big cities. The larger ones have flat roofs made with wooden beams, the smaller ones are domed and entirely of mudbrick. The low strength of the material limits mudbrick domes to small spans. It seems likely that the same construction was used in Antiquity because many of the tombs in the Valley of the Kings cut into the solid rock, which model the life style of the people buried there, have flat roofs for the larger rooms, but domed roofs for the smaller rooms.

So why didn't the Egyptians use arches for their stone structures to avoid the use of a veritable forest of columns in the interior? Temples were originally oftimber, which was considered a building material superior to mudbrick. One can see that from the decoration of the stone columns in both Ancient Egypt and Ancient Greece, and even more clearly from some early stone structures, whose ceiling soffit is carved to look like round logs of timber; the processional corridor of Pharaoh Sozer's Step Pyramid at Sakhara is an example of a very early Egyptian stone structure (about 2800 BC), whose stone ceiling is carved to imitate round timber logs. So the early stone construction was based on the existing timber technology, and not on the more appropriate technology used for mudbrick.

It was left to the Romans to take this logical step. From the stone arch and barrel vault they developed the cross vault and the dome, and achieved an enormous increase in span. The Pantheon in Rome, built in the 2nd century AD retained the record for span for 16 centuries, until long-spanning iron structures were developed.

The Roman invention of the masonry arch was the basis of most of the great architectural structures prior to the Industrial Revolution. The Byzantines developed the shallow dome, which greatly reduced the hoop tension that develops in the lower portions of a hemispherical dome. This was adopted and greatly enhanced by the designers of Islamic domes, who added the muquamas (or stalactite structure) to the repertoire.

The cross vault and its imaginative variations are quite properly credited to the Gothic masterbuilders, but the concept was Roman. The Romans preferred domes to cross vaults, but they built enough for the idea to have come down to the medieval masons. For

Figure 1
Storage sheds built from sun-dried brick to store materials and tools for the construction of the mortuary temple of Ramses II, around 1230 BC. These temporary buildings would ordinarily have been demolished on the completion of the temple, but this was left unfinished when Ramses II died. This proves that the Egyptians knew the arch a thousand years before the Romans made it the main form of durable long-span construction. It is difficult to say why the newly developed stone construction in Egypt imitated timber technology rather than the more appropriate sun-dried brick construction. Presumably the prestigious stone construction was based on the prestigious timber construction rather than lowly mud brick

example, the cross vault of the Basilica of Maxentius, which was prominently located in the Roman Forum, did not collapse until 1349 (due to an earthquake).

Brunelleschi was probably the first to use timber and iron reinforcement in domes to absorb the hoop tension and produce a lighter structure, and many Renaissance domes surpassed those of Ancient Rome in elegance, but not in span.

The Roman structural influence extended eventually as far as India, but probably not to Chipa, where the masonry arch was also discovered, presumably independently. The Chinese built a number of stone arch bridges, whose maximum span was only slightly less than that of the longest Roman masonry arch bridges; but the Chinese never attempted to build arched masonry buildings. Significant buildings in China, Japan and Korea were general ly of timber. In Japan some important wooden temples were rebuilt at regular intervals, according to tradition as copies of the original design. In China they were also replaced, according to tradition, by copies when they were destroyed or damaged by fire. Thus the designs may be old, but the material rarely is.

Iron eventually solved the problem of the structural deficiencies of timber and masonry, because it was incombustible and had tensile strength. The Chinese developed the manufacture of cast iron in the 6th century BC, but never used it as a major building material. The Europeans discovered cast iron only 1800 years later, and the material remained too scarce and expensive to be used as a principal building material until the early 19th century. The progress to the mass production of wrought iron and steel was then rapid, and their availability dramatically increased the potential for long-span construction. The largest domes were used for sporting events, and in the later part of the twentieth century their span had become so big that

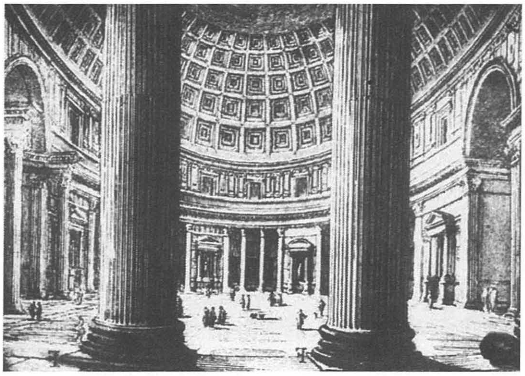

Figure 2
The Pantheon in Rome, around 120 AD, the longest spanning masonry structure prior to the mid-19th century. This is an etching by Piranesi

the people in the rear seats could not see properly what was happening at the centre, so that television monitors had to be installed. The maximum useful span had evidently been reached.

However, steel did not just extend the useful span of traditional construction. There was one thing it could do that had been difficult with traditional building materials: it could resist direct tension. This was particularly useful for bridges, and the longest bridges in the world today are suspension bridges. However, the suspension structure does not work well for buildings. There is no need for a building span the length of the Golden Gate Bridge, and the shorter spans can generally be handled better by less flexible structures, which are easier to make watertight and cost less.

The use of cable stays for buildings is, I think, an innovation made after the Second World War.They are

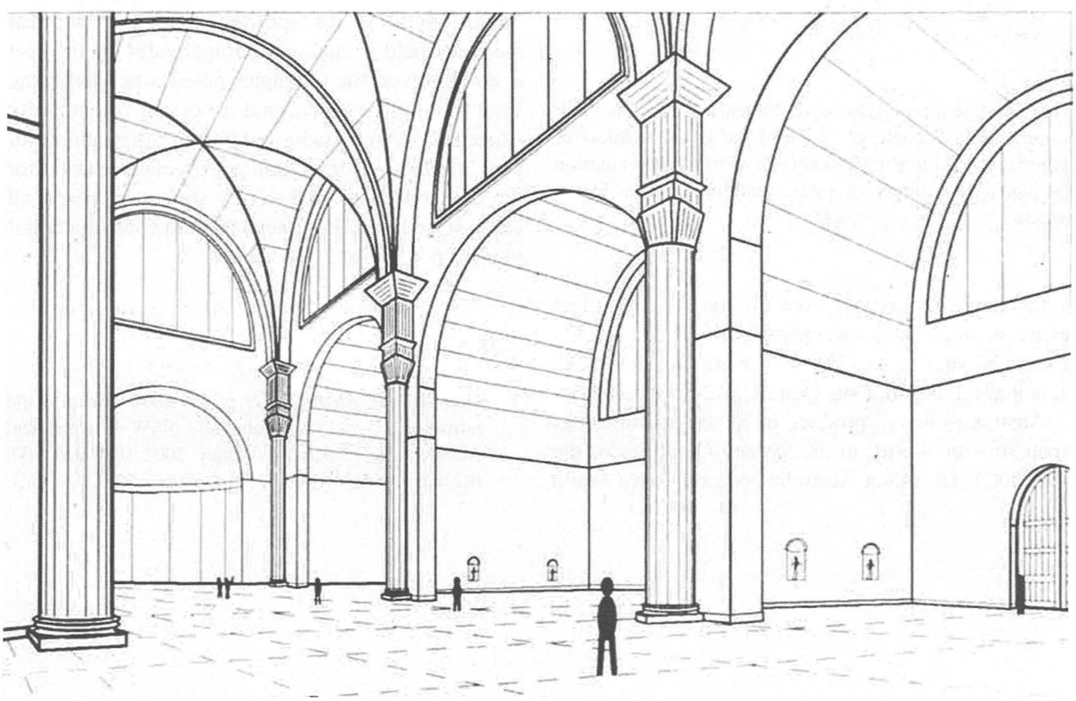

Figure 3
Reconstruction of the Basilica of Maxentius on the Roman Forum, around 310 AD, which must have been one of the most impressive vaulted spaces of all time, probably exceeded only by St. Peter's Basilica in Rome. It is now a ruin, but this reconstruction by the Roman Antiquities Service is probably quite accurate, as the roof collapsed only in 1349 during an earthquake

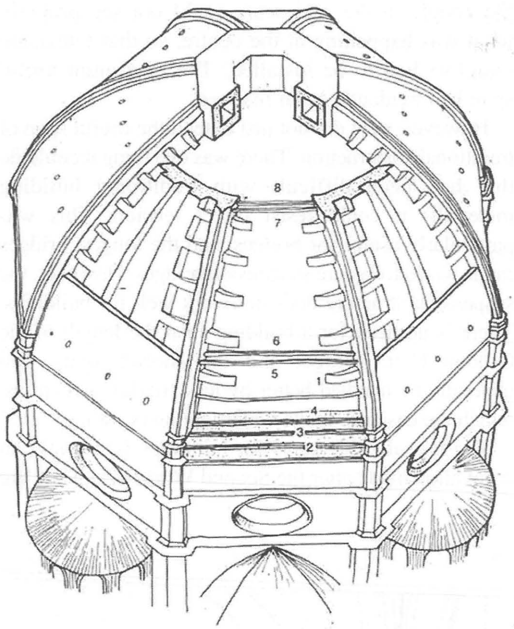

Figure 4
The structure of the Dome of Florence Cathedral, built around 1450. Brunelleschi avoided the great thickness of material needed in Roman domes to resist the hoop tension by placing a number of timber and iron chains in the structure

in fact the realisation of a dream of the designers of an earlier era for a sky hook, the replacement of a column under the roof by a cable above it. This has great potential for elegant long-span lightweight structures.

Australia did not produce many significant long-span structures prior to the Sydney Olympics of the year 2000. The major Australian cities all have a mild climate. Thus we did not need to provide long-span indoor facilities for sporting events or political meetings held at sub-zero temperatures or in great heat. However, the Olympics posed a new problem, because the international sporting community expected covered stadia holding a large number of people. Many of the buildings especially erected for the Olympics have noteworthy structures, nearly all cable-stayed, and I commend them to your attention if you are not familiar with them.[1]

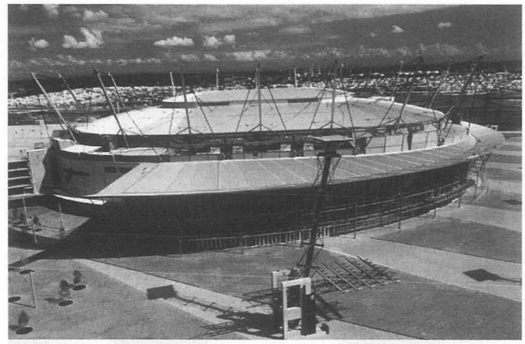

Figure 5
The Superdome built in 1999 for the Sydney Olympics. Its light construction is due to the use of cable stays, as in most of the long-spanning Olympic structures

Notes

1 *Monumental Australian Architecture*, selected and introduced by Chris Johnson, NSW Government Architect. BT Latitude, Sydney 2002 (PO Box 837, Blacktown NSW 2148).

The ancient approach to construction and the modern project

Salvatore D'Agostino

The Centro di Ingegneria per i Beni Culturali of the University of Naples «Federico II» has been carrying out research for over ten years in the fields of engineering concerned with the conservation of historical buildings.

Particular attention has been paid to analysing the ancients' practice of architecture, in order to understand their approach to construction. This has enabled us to draw up guidelines for the conservation of a heritage which, when monuments are considered as documents, is seen as a vast archive of material history. This holds good for our own heritage and also for that of other civilizations.

Our research has thrown light on the methodological differences distinguishing the concepts of construction and structure, construction being identified with the organism in its entirety, and structure considered as the supporting or load-bearing entity of the construction. This dichotomy lies at the heart of a global approach to the practice of architecture.

This reseach project aims to investigate the methodological and conceptual differences between the ancients' building practice and modern architecture. It highlights the unitary nature of that practice, which catered for a number of requisites; statics was one of these, but it was not given undue importance. Constructions were viewed in a much more complex spatial dimension than the schematic structural approach taken today. They were based on natural materials and techniques which recognised the importance of the art of building, ensuring a longevity measured in centuries, immune to the most common natural hazards such as seismic phenomena.

Building was undertaken according to «rules of the art» which remained unwritten right down to the 19th century, when some account of them was given in encyclopedias on the art of construction.

Today our entire building heritage needs an «archaeological» approach, for it belongs to a unique and unreproduceable building culture based on the laws of nature and experimental physics. The mechanical reinterpretation to which it has been subjected over the last century has resulted in a profoundly misguided approach to conservation. Ignoring every principle of correct historical evaluation, interventions were carried out using approaches, materials and techniques carried over from the new structural engineering, developed for industrial materials and based on the dichotomy between construction and structure.

The ancient construction

Since earliest times human society has used the materials provided by nature to build structures destined for public and private living. The structures that characterised villages and cities were modelled on the forms of nature herself. They could be

embellished and indeed made magnificent through the skills men acquired in working in wood and stone. Wherever «noble» materials were not available, human ingenuity was capable of adapting the same models to make similar structures out of mud and sand.

This process underwent a significant impetus when the natural forms were rationalised in the discipline of geometry. Architecture became the art of organizing man-made spaces. Human civilization has progressed as people have learnt to inhabit and construct, and the evidence of this progress represents essential documentation for our knowledge of history. The historical building heritage, in its dual valence as monument and document, presents a stratification over the millennia, offering itself to us today as an exhaustive archive of the material history of humanity (Bellomo and D'Agostino 2001).

Ancient builders were solidly rooted in their territory, and this gave them a profound knowledge of the environment in which they operated. The monuments which have withstood the test of time are the most telling proof of their creators' ability to cope with the various local hazards. Material expertise, linked to a familiarity with natural forms resulting in «specimen construction prototypes», has been handed down over the centuries in what we can identify as the «rules of the art» (Conforto and D'Agostino 2001).

Thus we see that an example of ancient building is a self-sufficient product conceived according to a global spatial approach in which the «structural» function is only one of the functions involved. Very soon buildings were made out of stone and mortar, and at that point they were conceived as «monuments of civilization» of infinite durability.

When building in masonry, the material is at once load and load-bearing, so that the architectonic and structural approaches form part of a single vision. This overlapping of the geometrical and structural functions permits a favourable distribution of tensions, which in any case are in general not particularly great. The physical and mechanical properties of masonry have always been part of the culture of construction, and the volumetries and construction prototypes never failed to consider the different nature of materials. The marked capacity of masonry for resistance to compression led to construction forms conceived for a naturally stabilising gravity, while awareness of a much lower resistance to traction emerges clearly in all the ancients' practical philosophy of building, witness the intercolumns, stone frameworks, Roman substructions, domes, large staircases and overarching structures (D'Agostino 1997).

The ancient construction was conceived as a single unit based on an expert synthesis of geometric forms and made of natural indigenous materials or else in brickwork, for humans have modelled clay since earliest times. The whole course of ancient architecture is characterised by buildings made out of an infinite variety of materials and geometric forms, and is as remarkable as it is many faceted. The different materials and geometric forms were manipulated by means of a range of building techniques which made an essential contribution to the final effect of each architectonic creation.

The Rules of Art as a practical philosophy

For millennia ancient buildings were conceived and erected without recourse to a mechanical-analytic vision of nature, but thanks to an ongoing reflection on phenomena leading to a comprehension of those «laws of nature» which were to be interpreted and stated in rational and analytic terms much later by the New Science. This process covered all human activities related to Nature, including agriculture, astronomy, and eventually also the study of the human body. This mysterious entity, as revered as it was left unexplored, has only recently been studied objectively and systematically, achieving extraordinary results in a very short space of time.

Thus for millennia the history of humankind was engaged in ongoing reflection and speculation on experience leading to the definition of «rules of thumb» which accompanied the progress of the various civilizations. These rules represent a constant recourse to praxis, and in fact they still inform our everyday activities, even when we are using the most sophisticated products of technological innovation.

For thousands of years architecture relied on «Rules of the Art» known to the architect who conceived the design and which were also familiar in a material, concrete form to the master builders. The results of this venerable culture are before the eyes of us all, but to trace an ideal trajectory we can cite the Baths of Caracalla in Rome, with its walls rising to a

height of some 50 metres erected in a matter of four years, the Royal Palace of Vanvitelli in Caserta, or again the Royal Palace in Madrid. These creations, spanning some two thousand years, are evidence of a building process whose underlying conception is in practice a mystery to us modern architects and engineers.

In 1870 l'Editore Dunod published the «Pratique de l'art de construire» by J. Claudel and L. Laroque, which includes a range of information about types of masonry: how they are to be put up, using which materials, for what purposes, with numerous empirical formulae for their proportions . In our opinion a knowledge of the building process adopted in antiquity is one of the key issues in current scientific research into the science of construction.

There is no doubt that the «rules of the art» were based on a spatial conception of the construction elements and not on the prevalently «monodimensional» outlook that characterises the New Science. The very definition of a construction element, formulated at the start of the 19th century, has come to have a schematic connotation due to its rational formulation. Furthermore it was perfectly evident, and it still is for those who operate in the field, that the «rules of the art» were formulated for specific contexts and were valid for a precise range of application; extrapolating them beyond this range is wholly inappropriate.

The fundamental construction element in modern architecture is the solid monodimensional beam or pile (upright or transverse), whereas in Roman architecture, for example, it was the substruction: a spatial construction with two elements in masonry, which may have been enclosed at the rear by a third element, completed above by a block with a flat extrados and circular intrados. These elements could be flanked and built over to form the façade of a building, or arranged in a horseshoe to make an amphitheatre, or superimposed as terracing to provide a massive supporting structure, as we see on the Palatine Hill, or again used in bathing establishments, villas, bridges and acqueducts. By studying this element we can identify the fundamental rule of art underlying its construction, and discover to what extent its rational conception embodied what we would call high safety coefficients, across the range of different applications. The only condition was that the basic length of the substruction, the «module», should not exceed a maximum, which we can quantify in six to eight metres Conforto and D'Agostino 1995).

This is the starting-point for a study of Palladio's designs for wooden bridges, for example, which were obviously limited by the construction techniques for overarching structures and by the technology for constructing in wood available in the 16th century. Similarly for his stone arched bridges, which imitated the Roman constructions (Di Pasquale 1996); current expertise did not extend to spans of 50 or 30 metres, but as long as the span was kept within five to 20–25 metres, depending in part on the type of stone used and on the context, the rules of art were valid, as we can see in any number of ancient bridges, such as Pont Neuf in Paris, dating from the end of the 16th century.

The so-called «builder's rule», which converted into practical terms the calculation of the metal beams required for roofing throughout the 19th century, could obviously not be extrapolated *ad infintum*, but was nonetheless perfectly valid for the usual modules employed in nineteenth century building.

Thus the attempt to identify the rules of the building art seems to be the key to reaching a full understanding of the historical approach to construction.

The Science of Constructions

The Science of Constructions is a recent Italian neologism for the discipline of mechanics applied to constructions, a useful synthesis of the Anglo-Saxon concepts of Strength of Materials and Structural Analysis. Throughout the progress of science over the last four centuries, the resistance of materials and the development of theories that extended the laws of rational mechanics to malleable bodies have always been at the centre of attention. Galileo himself investigated the problem of elasticity, although this was not to receive a definitive solution until 1856 with Barré de Saint Venant's «Mémoire sur la Flexion».

We do not intend here to rehearse the historical development of the Science of Constructions, admirably outlined in its theoretical attributes by E. Benvenuto (1981). We only wish to point out the scientific interest which surrounded problems of the

strength of materials for centuries, a field which saw many important achievements, ordered and systematised in 1826 in the famous lectures of Navier. These can be identified as the genesis of an autonomous science which adopts a physical-analytic model to interpret the structural behaviour of materials. Throughout the 19th century this new science developed and became systematic, often by considering significant problems of construction, as in the work of Jourawski and Castigliano. For alongside the analytic rigour of scientists such as Eulero, Lagrange and Cauchy, other researchers took a keen interest in problems posed by construction, which continued to be practised much as it always had been, with few substantial modifications.

As early as 1729 Belidor in his «Science des Ingénieurs» insisted on maintaining links with the practice of building, attempting a problematic synthesis between theory and technique which won great acclaim. In 1755 Gauthey, Soufflot and Rondelet carried out experimental research prior to building the French Pantheon, in order to determine the minimum dimensions compatible with the function, initiating the concept of safety coefficient. We should also record in this context the Italian Vincenzo Lamberti, the author in 1781 of «Statics of buildings with an exposé of the theoretical and practical precepts to be observed in erecting buildings in the interests of their durability».

At the turn of the 19th century there was a lot of interest in building in cast-iron and iron, which was initially adopted without an adequate theoretical background. The bridges over the Severn, built in the years 1776–79, and the Wear were constructed using blocks of cast-iron as if they had been blocks of stone. However, iron bridge building evolved rapidly, and both the material employed and the structural design were well suited to the Science of Constructions. By 1823 the time was ripe for the Parisian bridges of the Carrousel, by Polonceau, and the Invalides, by Navier, and the crowning achievement came in 1883 with the Brooklyn Bridge, comprising three mighty spans measuring respectively 284, 488 and 284 metres.

In parallel with this evolution, factories began to be constructed during the first decades of the 19th century using cast-iron girders. The construction typology of an external shell, an internal framework of cast-iron girders and wooden roofing became prevalent right across Europe. During the 1830s the familiar iron girders with the «H» cross-section began to be produced on an industrial scale, and this element soon became a constant in roofing, the stereotype of a girder which attained theoretical dignity in the work of De Saint Venant.

Meanwhile constructions in masonry continued by and large to adhere to traditional type, although there were some attempts at grafting in the new approach to construction, as in the typology formulated by the Italian Alessandro Antonelli, based on brick pillars, lowered arches with chains taking up the thrust and roofing with mini-vaults, all in brick.

The end of the 19th century saw a number of engineering manuals dedicating a lot of space to the strength of materials. From this moment onwards the Science of Constructions became widely applied in construction practice,[1] while scientific research came up with important results which made it possible to systematise the mathematical theory of elasticity, and struck out further in studying the ultraelastic and viscous behaviour of materials. This led to the theoretical definition of the structural behaviour of many construction elements viewed from a geometrical-mechanical perspective, such as arches, plates, slabs, shells, and so on. In addition complex problems of structural mechanics were tackled, such as the stability of equilibrium, which Eulero had already treated with great acumen. This voluminous mass of research, which is still on-going with specific connotations, gave rise, together with the introduction of industrial materials, to the greatest construction revolution of all time: the disappearance of constructions viewed as organic wholes and the advent of structures designed to fulfil merely a static function, in line with the mechanical model of the Science of Constructions, paving the way for of a new practice of architecture.

The construction revolution

The revolution which transformed the approach to building that had developed over many centuries was spearheaded by the Science of Constructions and the production of new industrial materials: iron and reinforced concrete, both able to be «scientifically modelled». This was in fact one aspect, of particular significance from our point of view, of a process of

radical mutation of the life and culture of humanity at large, produced by industrial development, social evolution and by a new socio-political dynamic affecting individuals that characterised the turn of the 20th century.

The new century ushered in a new phase for humankind, underpinned by dramatic progress in science and technology, so that from today's standpoint 19th century society appears almost archaic. The time-honoured approach to building continued to hold good throughout most of the 19th century, incorporating technological innovations that were marginal with respect to its fundamental principles. If we limit ourselves to the example of Italy, the last decades of the century saw the prevalence of Umbertine architecture, sumptuously deployed in many Italian cities. This construction typology, which over the years was toned down with regard to its decorative schemes, saw the gradual inclusion of elements first in cast iron and then in reinforced concrete, coming to be known as «of mixed structure», and continuing to characterise most building through to the end of the 1930s.

We need only cite three or four examples of the onset and affirmation of the revolution we have described: in Paris in 1889 Eiffel constructed the quintessential iron tower; in 1904 the same city witnessed a building in rue Franklin in reinforced concrete, the work of Perret, and shortly afterwards another situated in rue Reaumur in metal, built by Chedanne. In 1931 the towering mass of the Empire State Building rose up in New York, by which time the new architecture was ubiquitous, and Le Corbusier was perfecting his rationalistic approach. This marked the triumph of scientifically designed structures based on the model of the Science of the Constructions which inaugurated the construction industry involved in the production of both structural materials and an infinite range of «finishings», non-structural elements that completed the overall construction. The new architecture knew a golden age, well served by the new building techniques.

At this point the ancient approach to building, traditional materials and time-honoured techniques rapidly became obsolete, and were totally abandoned in academic and professional training and also in industry and the building trade. Roughly one century on from the revolution, we are surely justified in speaking of the material and construction rationale of the historical building heritage as «archaeological», in the sense that it is informed by a construction culture which has disappeared for good.

There is no need to illustrate here, even by highlights, the wonderful achievements of the new architecture and the scientific and technological development which has supported it and sometimes inspired it, in the realms of the construction industry, public works schemes and large-scale infrastructures. However, we must point out one negative aspect of this revolution which has been deleterious in both qualitative and quantitative terms. Since the middle of last century, following the end of the Second World War, an anonymous style of house building became prevalent, throughout the planet at large, which hemmed in the historical city centres, created vast, faceless suburbs and at times even whole new cities and industrial conglomerations, almost always in the absence of urban planning. Undoubtedly this process, which varied in quality from one country to another, was induced by great social mutations, new housing requirements and the industrial explosion that characterised the decades from the 1950s to the 1980s. But today we can perhaps recognise that this new building industry was the first great phenomenon of globalization, together with the spread of the motor vehicle. Alas these phenomena, which may well be inevitable, have produced a widespread homogeneisation, with serious consequences for the quality of life in historical city centres and the health of the natural environment.

We are already on the threshold of a new phase, with the «scientifically programmed invention» of new materials, beside which reinforced concrete seems an outdated, traditional material; and while the exploitation of property still favours vertical construction, we must now begin to consider different models of habitation based on communication and transport systems of the future.

THE NEW PROJECT OF ARCHITECTURE

The construction revolution has marked a definitive watershed between the historical building heritage and the new architecture, whose development and material configuration go hand in hand with the scientific and industrial approach to construction. The science of constructions was responsible for

elaborating the concept of structure, calculated according to the rigid norms which regulate design and execution. Scientific and technological process, based on the mathematical theory of elasticity, ensured the ubiquity of the module crossbeams-plus-pile, leaving architects to define the space and volume of a building and see to its functional completion with «finishings» added as adjuncts to the self-sufficient «structure». This new approach was made possible by the use of the new industrial materials, iron and reinforced concrete, in perfect synergy with the new structural concept.

This gave rise to a new project for architecture, in effect the mere synthesis of the compositional, structural, technological and assemblage components. These various components, which were bound to coexist and interact with each other, have developed their own autonomous processes in the sectors of architecture and engineering, favoured by an industrial boom which has created a mass production of components in continuous evolution. This has led to numerous sectorial specialisations, all contributing to the formulation of the new project. It seems to have been precisely the technical advances and the gradual acquisition on the part of the building industry of higher architectonic standards that have fostered that nondescript and frequently squalid style of building which has characterised, in all Europe at least, decades of post-war reconstruction, leaving its mark in the vast urban sprawls clutching at the historical centres of all our European cities.

Knowledge and Conservation of the Historical Heritage

The deep divide that has opened up between the ancient approach to building and the new architecture has brought about the dissipation of that material knowledge of building which over the millennia had characterised the evolution of world civilizations. In effect the entire art of construction known to our forefathers, with its rules of the art, materials and techniques, was abandoned in the space of a few decades both by the culture of architecture and by building practice. A whole field of professional craftsmanship, with a large number of specialisations, suddenly became obsolete. In spite of the fact that the latest trends in historiography have highlighted the great scientific significance of material culture, the history of architecture has gone its own way imperturbably, failing to recognise the value of a monument as document of material history. This fundamental failing has prejudiced both a scientific understanding of our architectonic heritage and its well-informed conservation.

The failure to recognise the intrinsic value of the ancient approach to building, with its traditional materials and techniques, and concentration on purely formal aspects has authorised architects, since the first decades of the 20th century, to adopt the materials and techniques of the new architecture in conserving the historical heritage, bringing about profound alterations in both its conception and its material conformation. The unequivocal sign that this technical operation lacked any historical or scientific grounding can be seen in its definition as «consolidation». From the Latin «cum solidus», this verb means to make sound, stable, long-lasting, to put in the condition of resisting, so that consolidation is an emergency operation intended to render a construction element stable.

It is hardly surprising that this technique of consolidation has parted company in practice both with the ancient approach to building and with the models of calculus that inform the new building techniques. A vast repertory of consolidation techniques was assembled in the mid 1970s, since when they have been assiduously reproduced in any number of textbooks and manuals. These techniques, ranging from wholesale injections of cement to metallic stapling and reinforced concrete splints, are in fact an invasive consolidation which profoundly alters the ancient building culture, without finding any justification in the models of calculus designed to interpret the static behaviour of the «new consolidated structures». A wide-ranging repertoire of cementification of Italy's patrimony of monuments can be consulted in the proceedings of AITEC 1981.

The other great «discovery» of the 1970s and 80s was the seismic vulnerability of the territory and the apparent inadequacy of the historical and archaeological heritage to withstand seismic phenomena: but far from being a «discovery», this is the lamentable outcome of a profound ignorance of history. This development has been made possible by (a) the total inability of historians of architecture to recognise in the ancient construction forms a pristine

document of material history; (b) the blinkered refusal of modern structural engineers to view the ancient heritage as an archive of the art of building just waiting to be opened up, able to disclose our predecessors' underlying approach and the rules of their art; (c) the ham-fisted approach of the building trade, willing to implement wholesale and highly profitable techniques of consolidation, in collusion with the technical and administrative institutions supposed to oversee them. It has been no easy matter to conduct a cultural campaign focusing on the respect of the monument as document, with the implications for materials and construction techniques, as the cornerstone for any intervention. Our message was that, far from being consolidated, the structure was to be restored respecting its construction rationale, materials and techniques, in order to «improve» that intrinsic potential for resistance which is compromised by degradation and damage inflicted by human action. In Italy the most notable results of this campaign in terms of legislation are to be seen in the notion of «improvement» introduced in norms on seismic events and the recognition of improvement as a principle of building restoration stipulated in art. 34 of the D.L. 29/10/99, n. 490 (Bellomo and D'Agostino 2000).

Unfortunately, the situation is much less healthy with respect to the static interventions still being carried out by firms and planners. Too many perverse financial interests, and excessive cultural inertia in terms of both planning and the so-called «restoration campaigns», are hampering a correct interpretation and application of the criterion of improvement. To this we must add the uncontrollable development of technologically advanced products produced by the building industry. More often than not the use of these materials represents an advanced form of consolidation which will bring about profound changes in the ancient structures without ensuring any reliable guarantees of durability and reversability.

Conclusions and new research perspectives

To promote the development and diffusion of a well-informed conservation of the historical building heritage, on a par with the restoration of works of art, it is essential to undertake a systematic study of the traditional approach to building, which will lead on to a full awareness of the material history of the practice of architecture.

This is the task for our historians, who should be able to call on a whole range of specialistic sectors in developing an archeometry of the building heritage. This will involve the systematic investigation of ancient forms, materials and cohesive substances, stratigraphic analysis and technological innovation in the interests of obtaining a more thoroughly scientific profile of historical structures. One field of particular importance for the analysis of ancient architecture is the identification of the rules of the art, involving the analysis of prototypes and manuals concerning the art of building; this is a specific task for structural engineering, charged with reconsidering these rules in the light of the scientific developments made by the science of constructions.

Finally it is indispensable to promote the training of professionals and entrepreneurs in the sector so that they abandon the deleterious practices of consolidation and implement a form of restoration which respects the underlying rationale of ancient buildings, their constituent materials and the techniques used in their construction. Since, as we have seen, the traditional art of building is an organic whole, restoration is an essential element which must be developed and discussed together with all the other aspects which, in a global vision, define the restoration project. An accurate anamnesis can help to return the monument either to its original configuration or to that emblematic form, of particular historical and artistic interest, which it has come to assume in the course of time.

Restoration must be based on the configuration of the building, defined in every aspect, and its static efficiency must be thoroughly evaluated. Only once these steps have been taken can «improvement» be undertaken, which, far from being mere static reinforcement with no concern for materials and techniques, involves the elimination of degradation and the correction or elimination of local instability using homogeneous and compatible materials and techniques. In sum we are talking about a «regenerative cure» for the whole organism, so as to pass it on to future generations with its value as a historical document intact in all its complexity.

Note

1. In Italy Alberto Castigliano published in 1884 the third volume of his *Manuale dell'Ingegnere*, Ed. Negro Torino, completely dedicated to the Resistance of Materials.

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Two traditional wooden building systems in Trentino (Italy)

Michela Dalprá
Antonio Frattari

Lying in the heart of the Alps, Trentino covers an area of 6 thousand 200 square kilometres. A thick hydrographic system crosses the Trentino and originates a complex system of valleys that are densely populated. Because for geographic position between northern Europe and the Mediterranean, Trentino has always been a land of encounter and exchange among the populations, as its history and interesting traditions show. An Alpine climate characterizes, nearly everywhere, the territory that is mostly a mountain landscape and it is covered with abundant forests where predominate broadleaf and conifers species that produce optimal timber. The Trentino landscape, so rich of orographic, hydrographical and material resources, has been an ideal scenery for the initiatives of the constructor man that was able to use the riches of the area and to colonize in the best possible way the territory.

The abundance of wood has favoured in the past a very great use of this material for the buildings construction and for buildings parts construction. In the traditional rural building, in fact, the wood, often is matched to the masonry. In Trentino the combination of masonry and wood in the buildings is variable according the areas and according to the building typology. In the traditional rural service buildings the masonry, generally, constitutes the basement, while the other floors and the roof structure are wooden. Traditionally, in many areas of the Trentino, the roof lining was wooden. The roofs were covered with wooden roofing-tile (shingles). Today the shingles are replaced improperly with tiles and sheet.

The constructive techniques used mainly for the construction of the traditional rural buildings, emerged from a research carried out in the Laboratory of Planning Building Design of the University of Trento in order to identify and classify the typologies of wooden buildings are the blockbau system and the framework system.

THE TRADITIONAL RURAL BUILDINGS IN TRENTINO

Specific typologic and constructive peculiarities characterize the traditional building of the Trentino. These peculiarities make different the Trentino's traditional rural heritage from that of other regions. In Trentino there are many wooden buildings and many buildings in wood and in masonry that meet dwelling and rural requirements. They have the function to shelter people, animals, tools, and to preserve agricultural products. In many areas these service rural buildings are separated from the building where the countrymen live. They are buildings independent, seasonal, placed at different heights according the activities to carry out. At the middle and high heights it's easy to find buildings with the cowshed on the ground floor and the barn on the first floor. Generally all these constructions are directed according the

ground slope: down there is the entrance to the cowshed, above there is the entrance to the barn.

Different are the names used in Trentino for the traditional rural service buildings divided into cowshed and barn. In the Giudicarie valleys (south-western Trentino) the buildings placed at the middle and high heights to shelter animals and to preserve the hay are called «ca'da mont». In the Fiemme and Fassa valleys (north-eastern Trentino) these buildings are called «tabià». In others areas like in the Sole valley and in the Rabbi valley (north-western Trentino) are called «masi». In some areas of the Trentino, in the Mocheni valley for example the name «maso» indicates a multi-floor building where the people live and work, where the dwelling and rural functions coexist. Here the «maso « is a family farm whit the house, the surrounding land and the agricultural equipment.

The materials used for the construction of the «ca' da mont», the «tabià» and of the «masi» are the wood and the stone. The mixed use of these two materials defines their different functionality. The stone was the more suitable material in order to construct the basement because it prevented that the wood touched the ground. The parts in dry or mortar masonry of the buildings were realized with natural local stone. The mortar was a mixture of sand, local earths and lime.

The wood was the optimal material for the barn in order to obtain a good aeration and in order to prevent the hay fermentation and its spontaneous combustion. The wood was easy to find in the rich forests of trees that there are in Trentino. The mainly used species were the larch (*Larix decidua*) and spruce (*Picea abies*). The larch is much resistant to the action of the atmospheric agents and it has good physico-mechanic characteristics. The spruce has characteristics similar to the larch, but it is not so resistant to the atmospheric agents than the larch. The spruce was preferred for inner structure and inner coating.

From a study on the typologies of the Trentino's rural buildings are emerged two constructive techniques for the construction of wooden buildings: the Blockbau system and the framework system. The framework system presents two variations: the «ritti e panconi» system and « a crociera» system.

The Blockbau system

The blockbau system is a typical constructive technique of the Alpine regions rural building at the middle and high heights. Mainly in the Fassa valley, Fiemme valley, Primiero's valleys, Rendena valley,

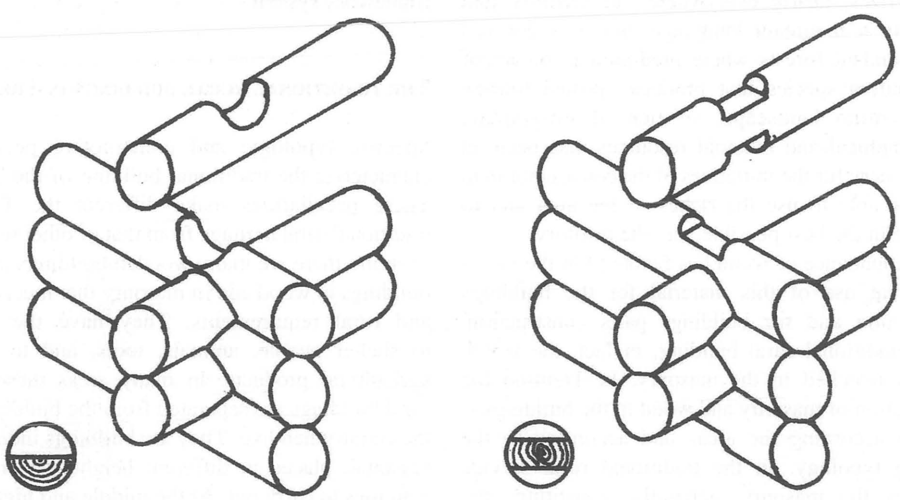

Figure 1
Different solutions of corner joint in the Blockbau system: a) the cut «a half-lap joint», b) removing of wood from both the upper and lower faces of the log.

Genova valleys and in Fersina valley, the wooden parts of the service rural buildings are realized with this constructive technique.

The walls are made of overlapping round logs, that cross in the corners. Two different solutions of corner joints are been used in Trentino. In the first solution «a half-lap joint» the upper half of the log section is removed. In the second solution the wood is removed from both the upper and lower faces of the log (Figure 1).

In order to obtain a better connection between the overlapping horizontal various elements and to stabilize the logs on the vertical plan, in Trentino have been used three different log blocking systems. They are:

— The system that entails the interposition between the logs of wooden dowels or pegs arranged at a fixed distance and staggered vertically one from other.
— The system that entails the introduction between the logs of wooden elements double wedge shaped. This is a system that on one side improves the logs stability and on other side improves the aeration in the rooms. In fact the wooden wedges outdistance more the logs one from the other, creating openings that favour the ventilation in the barns. In the rooms where it is important that the walls are airtight, watertight, the opening are sealed with pieces of musk or pieces of wool.
— The blocking system constituted from two vertical wooden elements that are put in the appropriate holes of a wooden element placed perpendicularly to a logs.

The second blocking system is widely diffused in the traditional rural buildings of the Fiemme, Fassa and Primiero's valleys in the eastern Trentino. The third blocking system is characteristic of Fiemme Valleys. In some «tabià» of Bellamonte has been used this blocking system.

The wooden floor that separates the stone masonry from the wooden barn is a made with wooden boards that rest on external walls and on beams supported from one or more pillars.

The roof of the buildings executed with the blockbau constructive technique is the pitched roof. The pitches are jutting. The roof, generally, has a wooden double roof frame. The large roof frame can be:

a) with purlins and rafters, b) with rafters on truss, c) with rafters on ridgepole. The ridgepole can be supported from a intermediate pillar and from pillars placed on the log walls. In some cases, in place of the ridgebeam, there is a longitudinal wall to support the rafters. In some areas, the ridgepole can be supported from pillars external to Blockbau walls that rest directly on the ground.

In the «ca' da mont» of the Chiese valley is interesting the constructive solution of external pillar blocking to blockbau wall through a curved wooden element inserted between the overlapping logs.

The roof lining traditionally was constituted from shingles, today in great part replaced from tiles and sheets. The shingles are placed on joists or on wooden planks. The shingles are wooden boards, 70 cm long and 10–15 cm wide. They are obtained by splitting block of larch or of spruce. In order to assure the roof lining impermeability the shingles are been placed on the small roof frame staggered and partially

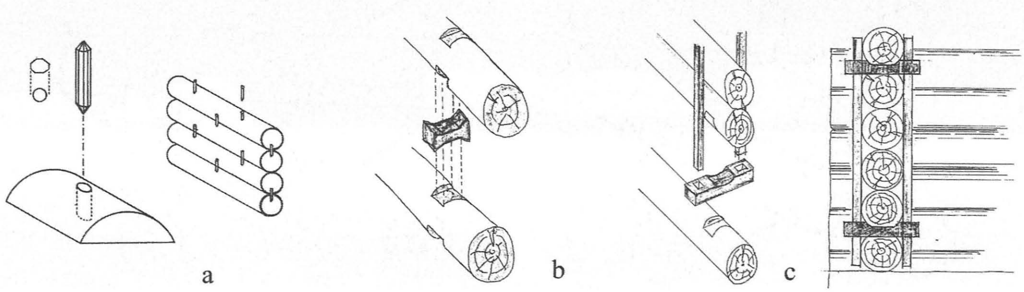

Figure 2
Log Blocking systems: a) system with wooden dowels or pegs, b) system wooden elements double wedge shaped, c) two vertical wooden elements put in the appropriate holes of a wooden element placed perpendicularly to a logs.

Figure 3
«Tabià» of Bellamonte in the Fiemme valley. Traditional rural building executed with the blockbau constructive techniques. The roof frame is made with purlins and rafters

overlapping. In the oldest constructions the shingles were simply laid on the small roof frame and were held in position by slim logs, overlapping at intervals and parallel to the eave line which were weighted down with very large stones. After the roof lining stability was obtained by nailing the shingles. The slope of shingles covered roof is the same in all Alpine regions, about 26°. With a higher slope of the roof the snow tends to remove the shingles, with a smaller slope of the roof the mantle loses the impermeability.

The door opening was created with two jambs that are correlated to the overlapping logs with tongue and groove joint. The cut log has the tongue, the jamb has the grove. The top of the jambs are fixed in the log that is lintel. The bottom of the jambs rests on the stone masonry basement.

The windows are not necessarily present in the rural service buildings *cowshed-barn*. In several cases the only openings are the doors. The windows that

Figure 4
«Ca' da mont « of Planezzo in the Chiese Valley. Constructive solution of external pillar blocking to blockbau wall through a curved wooden element inserted between the overlapping logs

illuminate the cowsheds, when present, are of small dimensions. These windows in the blockbau structures are practically a simple hole realized removing wood in two overlapping logs. The logs have in height a cutting depth of one radius (until the center of the transverse section).

The windows of the barn, when present, have greater dimensions. They are high from three to five logs. In order to prevent instability phenomena of the whole construction the windows have window-posts that stick in the logs with a tongue and groove joint.

Balconies, overhangs characterize the facades of many traditional rural buildings. In the Fiemme valley and in the Fassa valley there are some «tabià» that have the top floor with a considerable overhang. These overhangs are covered with wooden planks nailed to wooden structure and create other spaces in order to dry the hay and to store agricultural products (Figure 5). In some «cà da mont» in the Chiese valley the cowshed's doors are not on the façade. A space subdivided in two parts called «cetine» and covered with wooden vertical planks moves the doors back.

The framework system

The framework system is widely diffused in the Trentino's valleys. This constructive technique placed side by side and in some cases replaced the Blockbau system from 1400–1500 with the saw-mills construction and with the consequent possibility to cut and saw the logs and to produce planks for the coverings. The walls in the Trentino's framework buildings can be made with the «ritti e panconi» system or with the «crociera» system.

The «ritti and panconi» system is diffused mostly in the south-western Trentino, for example in the Chiese valley (Figure 6). The walls are realized placing vertical elements (ritti) to regular step on the building mesh. The vertical elements have laterally grooves in which are inserted horizontal elements that are placed one on the other. The horizontal elements, called «panconi», are large boards. In order to insert the large boards in the uprights grooves the end of the large boards is worked with the axe. The working irregularity on the long part of the large boards prevents that the large boards fit together properly. In this way there are slits between the large boards that favour the ventilation.

The «ritti and panconi» system could be considered the evolution of the blockbau system concerning the connection between vertical element (ritto) and horizontal element (pancone). The large boards are overlapping like the Blockbau logs, but in this system they don't have a supporting function. In the «ritti and panconi» system the uprights and the beams have the principal supporting function. The large boards plug the walls.

The uprights are high like a floor, they are placed between the upper surface of the inferior floor beam

Figure 5
«Tabià» of Penia in the Fassa valley. Traditional rural building with wooden part in blockbau system. The top floor has overhang covered with wooden vertical planks

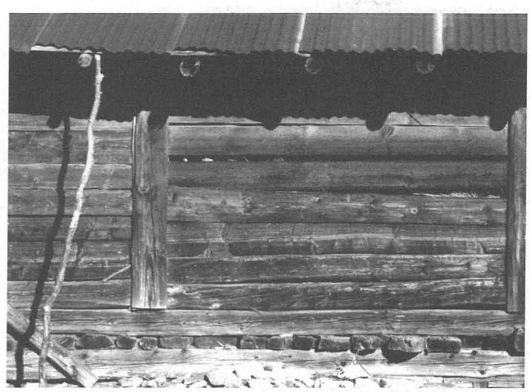

Figure 6
«Ca'da mont» of Planezzo in the Chiese valley. Building with wooden part in blockbau system. Traditional rural building with wooden part in frame work «ritti and panconi» system

and the lower surface of the above floor beam. The uprights joint to the beams through a wooden dowel that is fixed in the head of the upright. Sometimes wooden diagonal braces are fixed to the uprights with a simple overlap.

The framesystem called «a crociera» is mainly diffused in the north-western Trentino in the Rabbi valley and in the Sole valley. In this system the walls are realized with squared wooden elements of small dimensions that constitute the frame (posts and beams) and with planks in order to close the walls. The vertical elements in order to support the peak load are connected to the beams with wooden braces (Figure 7). The braces are fixed to the frame with dowels. The correlation between the brace and the upright is a joint: the brace and the upright are worked in order to penetrate one into the other. The correlation between the brace and the beam in a simple overlap with nails. The form of the braces is generally rectilinear.

The planks in order to close the walls are fixed on the inside of the frame elements in a horizontal position or in vertical position inside. The planks are generally larch. In some areas of Trentino, in the Rabbi valley for example, the vertical planks are placed in slides obtained with two spaced small ledges that are fixed to the beam. This solution make easy the assembly phase and the planks substitution and in this way the planks can move because the arrangement movements without the risk of cracking.

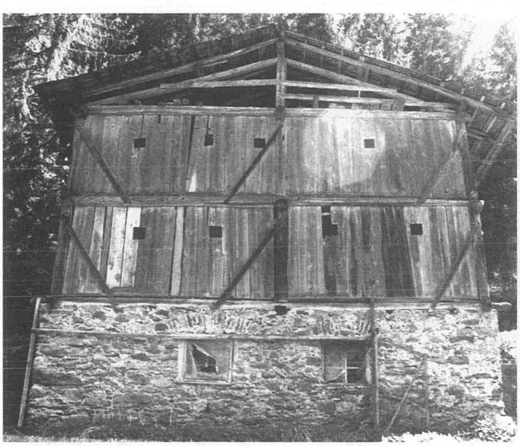

Figure 7
«Maso» in the Rabbi valley. Traditional rural building with wooden part in framework system with wooden braces

In the framework rural buildings the openings for windows have small dimensions. In the covering planks of the barn there are holes for the ventilation. The holes that have recurrent geometric figures that take on, also, a decorative character. In order to increase the ventilation in the barns in place of the small holes, sometimes in the planks are made vertical openings on the every plank.

Generally in the framework buildings the roof is a gable roof. Ledges on rafters support the roof lining. The rafters rest on the ridge and on the purlins. Wooden pillars hold up the ridge. The roof structure can be constitute from wooden truss. The traditional roof lining is made of wooden shingles. The gutter is a hollowed half-trunk of larch. Wooden bracket hold up the gutter.

The gable can be open, or closed with wooden planks or wooden ledges placed so that obtain a large variety of solutions that to characterise the façade.

A lot of framework buildings have wooden balconies that are drying-structures. Generally they are at the first level, but we can find examples of wooden balconies at the first and the second floor along all sides of the building.

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The formal unity of the Greek Temple: The realization of the 9/A capital replica of Apollo Epicurius' Temple at Bassai

Claudio D'Amato Guerrieri

In recent years there has been a growing interest in preserving both urban heritage and the European landscape. The need to repair damage to historical monuments of the European tradition while maintaining the particular qualities of local areas is becoming increasingly evident.

In many cases the urgency of reconstruction has often resulted in the creation of built fabrics and architectural details, characterized by a low design and construction quality.

The possibilities offered today by informatics' technologies (i.e. cad/cam processes), open new horizons within the theoretical debate on restoration and conservation, in terms of both partial and total reconstructions, architectural text integrations (substitution of damaged elements), or integral reproduction of original ruined architectures.

However this specific approach to restoration of monuments of the European tradition poses several delicate questions of both theoretical and practical nature, that have to be solved in the light of a generalized use of these methods as alternatives to existing ones for replica production.

The production of replicas in conformity with originals requires as essential preliminary condition the reconstruction of entire original design (i.e. original shapes and material choice), and therefore the conceptual and stylistic re-appropriation of the traditional construction processes.

In European countries stone represents a material so common in nature that in the past centuries the different civilizations largely employed this material in architecture.

The generalized use of load-bearing masonry systems with an organic character in traditional construction, contributed to the definition of the specificity and cohesion of this cultural area: stone architecture of the past have a strong identity, that results in a unified experience in which the synthesis of construction and design produces the architectural impression.

The Greek Temple could be easily considered as a paradigm in this sense: here stone, used as unique material for the definition of the aesthetic character, expresses directly the structural and decorative geometrical texture, and also defines the formal unity

Figure 1
View of the temple from Noth-West side bifore the restoration works (Eforia Z's Archives, Olympia)

of the architectural system as a whole. Moreover it is characterized by an exact design and perfection in execution, aspects that make it exemplar.

For this reason the damaged original 9/A Doric capital of Apollo Epicurius' Temple at Bassae, has been chosen for the elaboration of cad/cam processing software, and associated 3D virtual simulations for its realization with a CNC machining center.

The development of stereotomy techniques using modern advanced technologies will support the cultural heritage of each European region, and will open new perspectives for the design of architectural elements, providing competitive solutions in terms of quality, speediness of realization in comparison with manual fabrication. They will also help the traditional role of arts and crafts, but also that of stereotomy, to evolve.

The Replica of the 9/A capital of Apollo Epicurus' Temple at Bassae

The work presented here resulted from an ongoing research project[1] on the experimentation of cad/cam procedures to the realization of architectural elements in freestone, to be laid dry.

This research project, is focused not only to the ex novo designing, but also to monuments' restoration.

The research's applicative object is a very restricted one (with no underlying hypotheses for the integral reconstruction of monuments, although this is not excluded theoretically). Moreover its objective is not that of taking a stance in any ideological dispute over whether it is correct to integrate, by remaking them, stone elements that are missing or damaged within the context of restoration; but only to demonstrate that it is possible to produce them satisfactorily using modern computerized technologies.

This is a possibility whose premises consist on the one hand of a knowledge of the original project, not merely of the individual element to be remade but obviously of the entire building; and on the other, of the capacity for attaining perfect workmanship. Obviously such workmanship cannot be that of a craftsman, whose skill, however great but always dependent on traditional tools and techniques, would never be able to reproduce the material conditions in which the original piece was created.

When these two preliminary conditions, deemed indispensable for any procedure of remaking architectural elements in freestone, are satisfied, such elements can be called true «replicas»[2].

Among the possible applications, it was decided to operate taking as model the Doric capital, for its capacity to clearly demonstrate the thesis, since the apparent simplicity of its form tolerates no imperfection in design or execution.

The concrete case under study, coming from the scientific collaboration between the Dipartimento ICAR at the Politecnico di Bari and Eforia Z of Olympia on the one hand, and on the other CMS of Zogno (Bergamo), worldwide leader in the design and construction of numeric control machines, concerns the remaking of capital 9A from the temple of Apollo Epicurus at Bassae.

In agreement with the Commission for restoration of the temple, headed by the archaeologist Joannis Tzedakis, it was decided to proceed experimentally to the realization of capital 9A, integral in conformation, by electronic means; and contemporarily to the manual realization of capital B3, to be replaced. The purpose was that of launching a vast process of comparing the results, the times required for realization and the production costs.

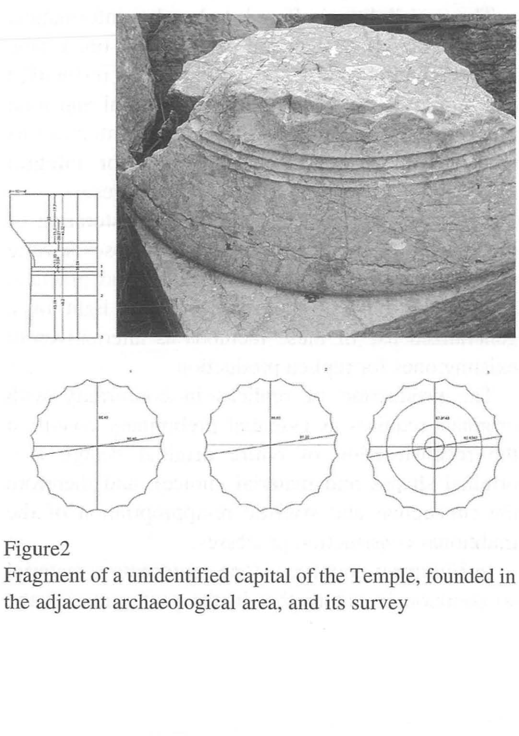

Figure2

Fragment of a unidentified capital of the Temple, founded in the adjacent archaeological area, and its survey

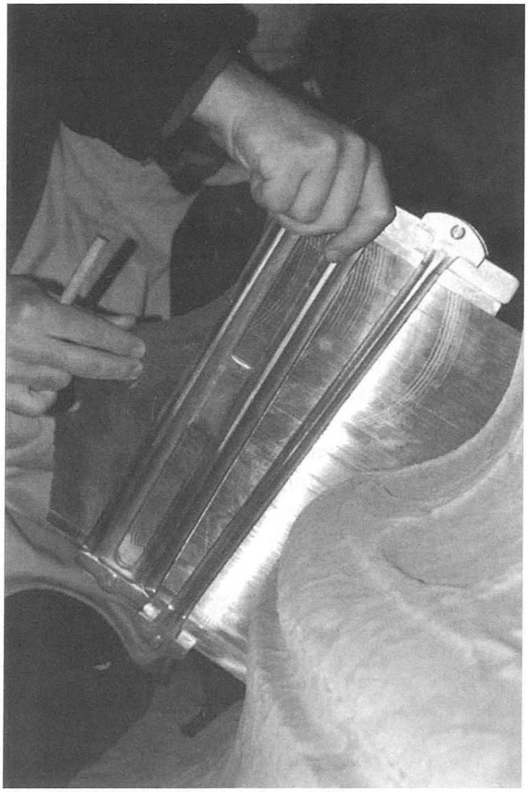

Figure 3
Base survey of the Ionic order of the cell using a outliner (architects: V. Cascione and G. Fallacara).

The experimental nature of the operation allowed us to:

— select as material Carrara marble, suitable for its degree of hardness to the technical characteristics of the set of tools (milling cutter and tips) available at the time of realization;
— produce a half capital, in consideration of its weight (approx. 1500 kg for a mean size of approx. 100x 60x120).

The time required for study (defining the laws of composition and examination) and designing the three-dimensional model was approximately 30 days (April 2000); the time required for realization, about 5 days (September 2000).

Characteristics of the Apollo Epicurus' Temple at Bassae

The Apollo Epicurus' Temple at Bassae presents elements of both innovation and tradition, audacity and control, research and calculation, contrasting factors which concur to define the overall quality and the high artistic expression of the building.

Tradition attributes its design to Ictinus (for a non-specialist approach to the problem, see the popular book by Rhys Carpenter, 1976).

The temple constituted a prototype that spread in the following century beyond the geographical limits of the Peloponnesus, extending up to the boundaries of the Greek world.

As concerns the exceptional nature of this temple W. B. Dinsmoor writes: «I believe we can say that within the perimeter of the peristyle may be found more fascinating problems than in any other building in the world of ancient Greece»[3].

The temple is peripteral-hexastyle (6 x 15 columns) distinguished by the contemporary presence of the Doric order on the outside, in the peristyle, the pronaos and the opisthodome; and Ionic and Corinthian orders on the inside. Ionic are the columns standing against the spurs, which articulate the interior space of the cell and the frieze at the top, while the column placed on the axis of the cell is instead Corinthian.

The proportions of the layout, very elongated (width/length ratio: 2:47) confer on it archaic features, in contrast to the «short» layout of the 4th century temples. This elongation is due also to the existence of an «adyton» between the opisthodome and the cell proper, presumed to be the original place of cult which, thanks to the presence of the eastern door of the cell, would have allowed the statue of the god to be placed facing the rising sun, as in a temple oriented in canonical manner.

As regards the Doric order, the columns in the peristyle, made of local calcareous rock, have twenty flutes separated by a very fine relief on the outside (2 millimeters) and a wider relief in the pronaos and in the opisthodome (about 4 millimeters). W. B. Dinsmoor has observed that the columns were wider on the facade and that the intercolumnation on the facade is wider than on the sides.

As in archaic times, the top of the shaft is adorned by three horizontal flutings, called dactylis. As

regards the Doric capitals, at least four different categories are reported in the sources: those of the northern facade, those of the other three sides of the peristyle, those of the pronaos, and those of the opisthodome. The first two categories differ from each other in size; the capitals on the northern facade are wider and taller than those on the rest of the peristyle. The profiles of the capitals on the opisthodome do not differ greatly from those of the capitals on the facade. The only obvious difference lies in the manner in which the flutes end below the echinus, that is with a slight curve, not with an almost horizontal elliptical calotte as in the capitals on the northern facade.

Accordingly, should examination of the Doric capitals confirm W. B. Dinsmoor's thesis concerning the two stages of construction of the temple, the dates would have to be changed: the first stage to around 450–430; the second stage to the first decade of the 4th century. These dates, in fact, are consistent with the information deduced from the profiles.

Studies and surveys of the Temple at Bassae

The temple has been the subject of studies the results of which are contrasting. A comparative analysis of these results reveals differences in both objectives and investigational methods (surveys and classification).

Joachim Bocher, the French architect attributed with discovering the temple in 1765, made the first study drawings of the monument. These drawings, which remained unknown for over two centuries and were published only in 1968, were done in pen and ink. Those which have survived show the plan and elevation of the monument viewed from the side ideally restored and the facade, also graphically integrated.

A more thorough study was conducted in 1811 and 1812 by a group of fourteen amateur archaeologists, all scholars and artists of different nationalities (French, English, German, Baltic). They belonged to an association which they called «The Xenoi», whose members included Carl Haller von Hallerstein, director of excavations, Otto Magnus von Stakerberg, draftsman, Charles Robert Cockerell, who did not participate in the subsequent diggings, and John Foster.

The notes and drawings in Haller von Hallerstein's notebooks, executed during a campaign of excavations, represent one of the most reliable sources of information, as valuable as that of the temple itself, and are extraordinarily important for all those architectural elements which were later dispersed.

In his drawings Haller von Hallerstein made a scientific survey of all of the architectural elements, going so far as to define their regulatory geometries. For example, his drawings of the Corinthian capital (later destroyed) show the succession of the investigative stages, from an overall representation of the find up to an in-depth study of the generator plots.

Among the most recent studies of proven scientific worth, conducted starting with direct examination of

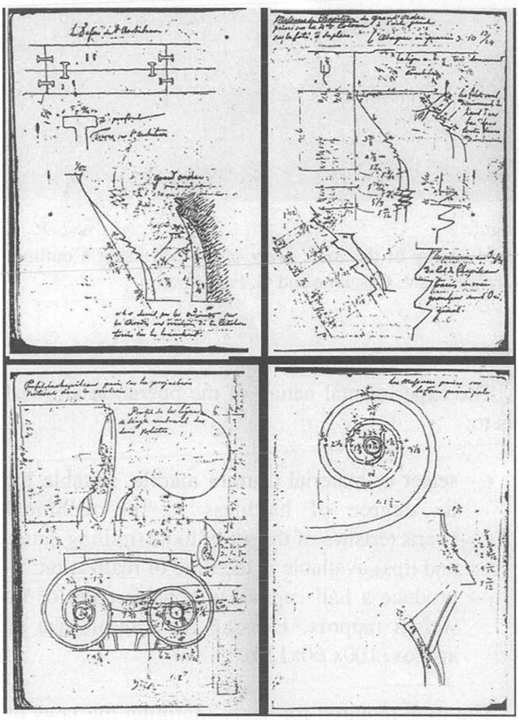

Figure 4
Karl Haller von Hallerstein, drawings from study notebook of Apollo Epicurius' Temple (from G. Roux, Karl Haller von Hallerstein, 1976)

the temple, are those of Lucy T. Shoe (1936) and of Friederick Cooper (1996). Their drawings reveal the different objectives pursued.

Starting from the 1920s the Greek Ministry of Culture has promoted a campaign of surveys, which today constitutes the most reliable scientific basis yet published (edited by D. Svolopoulos, 1995).

In the academic year 2000/2001 the Faculty of Architecture of the Bari Polytechnic Institute held a workshop dedicated to study of the temple (coordinator: prof. Claudio D'Amato; students: Mariangela Alicino, Francesca Aulicino, Cosima Carone, Francesca Cavone, Valeria Chieti, Giuseppe Dell'Aquila, Simona Dentico, Loredana Donatelli, Alessandra Paresce with the assistance of the graduate students Giuseppe Fallacara and Annalisa Di Roma). In agreement with Eforia Z of Olympus and in collaboration with the architect Sofoklis Alevridis of the Temple Commission, a campaign of surveys was carried out on April 9–12, 2001.

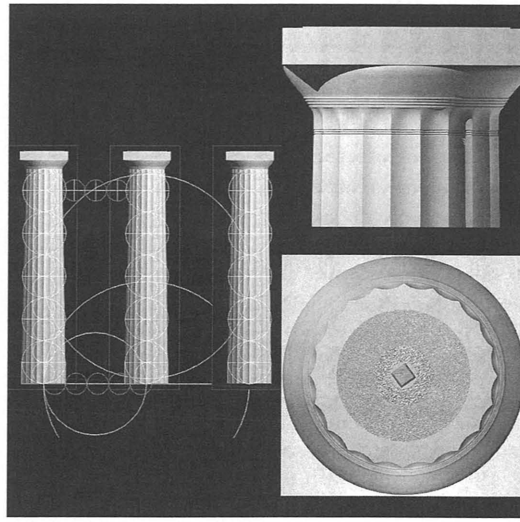

Figure 5
Politecnico di Bari, Dipartimento ICAR, Dottorato di ricerca in Progettazione architettonica per i Paesi del Mediterraneo: virtual reconstruction of the 9A column and studies on its proportioning; 9A capital's profile and horizontal sections

The modelling of 9A capital of the Apollo Epicurus' Temple at Bassae: structural geometries of the capital and computerized modelling techniques.

The «electronic» modelling of the capital and the eighth drum of column 9A was conducted by processing the data derived from traditional survey (investigation, examination?) (manual) on a scale of 1:1.

Capital 9A consists of a square-based parallelepiped abacus, an echinus whose contour is defined by a polycentric or spline curve, by a collarino and by the end of the fluting which is joined to the collarino through a complex surface.

The eighth drum 9A consists of a truncated cone with fluting and engraved end (hypotrachelion).

The identification of the different parts of the capital was done through computerized polygonal CAD modelling by «surfaces» and not by the parametrizing of «primitives» or «extrusion» of polygons, for the purpose of facilitating manipulation of the 3D object.

The abacus, a square-based parallelepiped volume, was obtained by the mutual and orthogonal composition of flat surfaces («2d faces»).

The echinus and the collarino, a single volume of rotation, are the result of a surface of revolution,

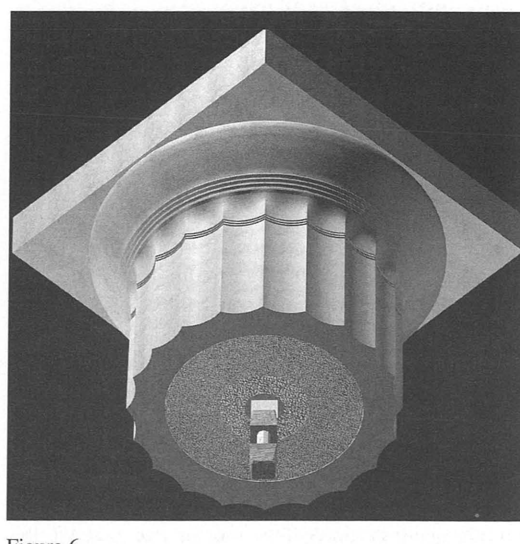

Figure 6
Politecnico di Bari, Dipartimento ICAR, Dottorato di ricerca in Progettazione architettonica per i Paesi del Mediterraneo: Axonometric view of the 9A capital virtual reconstruction

obtained by rotating the profile of these elements for 360° around an axis of radial symmetry.

The collarino, serving as union between the end of the fluting and the echinus, mathematically defined as quadric surface, was obtained through a «polar series» of a bilinear curved surface consisting of 20 elements through an angle of 360°. This surface was modelled in two successive stages: the first by interpolating a «Coon surface» for four spatial curves: the first of these is the portion of convex circumference termination of the annuli, contained in the horizontal plane included in an angle of 9° (half of 360°/20); the second is the profile of the fluting, contained in the vertical plane passing through the axis of radial symmetry; the third is the section measured at the centreline of the fluting, contained in the vertical plane passing through the axis of radial symmetry; the fourth is the portion of concave circumference at the base of the fluting contained in the horizontal plane. The surface determined in this way represents half of the global surface, which is completed, in the second stage, by «mirroring» the surface found according to the axis passing through the centre point of the fluting, perpendicular to the axis of radial symmetry.

The drum consists of a surface passing through three sections contained in horizontal and parallel planes, measured at different heights. The hypotrachelion has been obtained by generating a surface having as generatrix the raised profile (saw-tooth section) contained in the vertical plane passing through the axis of radial symmetry, and as directrix the section of the drum, contained in the horizontal plane, at that height with the fluting.

Today's processes of three-dimensional computerized modelling provide, to various degrees, a complete description of any real object, by reappropriating in virtual manner all of the mathematical-geometric laws which subtend the physical description of the object.

Through management/unitary appropriation of the geometric apparatus of the object under analysis, it is possible to investigate a multitude of phenomena at the basis of its conformation.

The forms possessed «in positive» restitute the importance of structural processes which can be hypothesized as «negative» (as in the case of the profile of the hypotrachelion and the generator tool).

The conformational unity of the computerized model supersedes processes of graphic representation

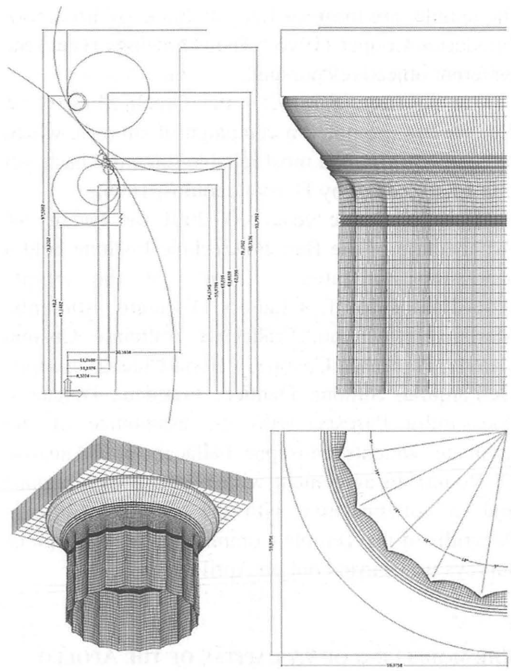

Figure7
Politecnico di Bari, Dipartimento ICAR, Dottorato di ricerca in Progettazione architettonica per i Paesi del Mediterraneo: Constructive geometries of 9A capital (PhD student: G. Fallacara).

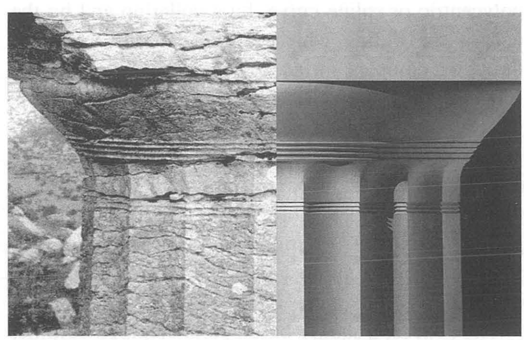

Figure 8
Politecnico di Bari, Dipartimento ICAR: comparison among the 9A capital actual conditions and its virtual reconstruction (arch. Vito Cascione).

by plane projections (which provide poor interpretations of reality), being at the same time both one and infinite (the planes tangent to the infinite points of a sphere at whose centre the object is placed being as many as can be imagined!) planes of graphic representation.

In other words, in possessing the computerized model we possess a reality which is the more real the more virtually it can be described.

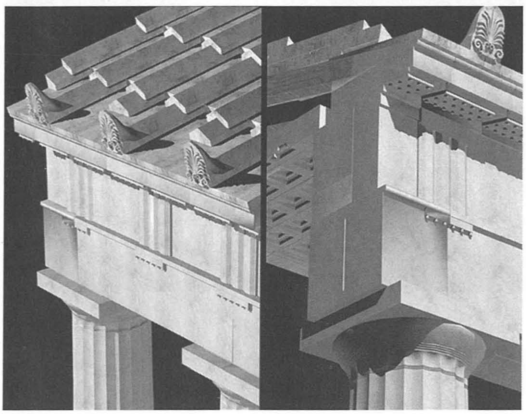

Figure 9
Politecnico di Bari, Dipartimento ICAR, Dottorato di ricerca in Progettazione architettonica per i Paesi del Mediterraneo: virtual reconstruction of the Doric order and of the temple roof (PhD student: G. Fallacara).

Stage of CAM engineering and NC post-processing of the file

The CAD/CAM software used for realization of the Capital is MasterCAM. By importing the model into the dwg format it has been possible to read the three-dimensional geometric conformation of the capital and to proceed to all subsequent steps[4], first among them the re-dimensioning and positioning of the surface area (to be machined) on the machine table.

The next step was selecting the milling cutters[5] suitable to the various parameters required by the program: geometric nature of the «object», type of machining (rough-finishing, finishing with 3/5 feed drives), material utilized (Carrara marble), speed of feed and rotation of the milling cutter. The type of movement (zigzag, spiral, etc.) to be used for removing the material, within a «pocket» of volume of the object, was then selected.

Having effected all of the CAM parametrizing, we then prepared the NC postscript file, directly readable by the controller of the 5 drive feed CMS-MAXIMA utilized for machining.

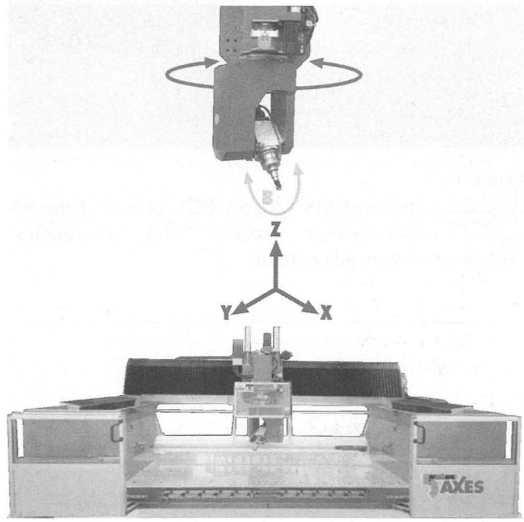

Figure 10
CNC Machining Centre of CMS, with 5 axes. Spindle's degrees of freedom in CMS Maxima machine.

Machining stages

It was deemed advisable to carry out preliminary rough-machining to eliminate the material of the blank that would have prevented the tool from approaching the surface of the piece.

The rough-machining stage called for the selection of the type of tool progress (zigzag, spiral, etc.) which the tool executed during machining, remaining orthogonal to the fixed table, and the choice of type of tool; in this case a cylindrical milling cutter was selected.

The next stage of work was finishing[6]. This process was carried out with 5 feed drives and with a spherical tool. During the finishing stage, the machining allowance (amount of rough material left above the actual surface) was removed, and machining of the details was carried out.

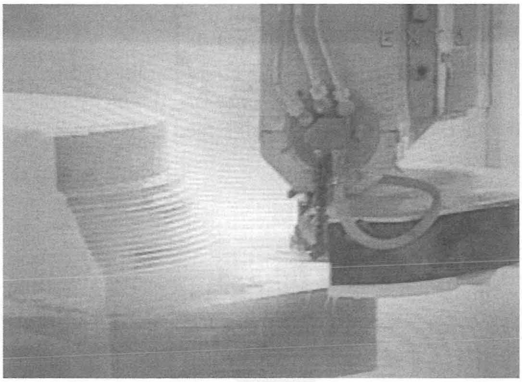

Figure 11
Politecnico di Bari-CMS (Zogno, BG): phases of the 9A capital reconstruction process with informatics' technologies (September 2000).

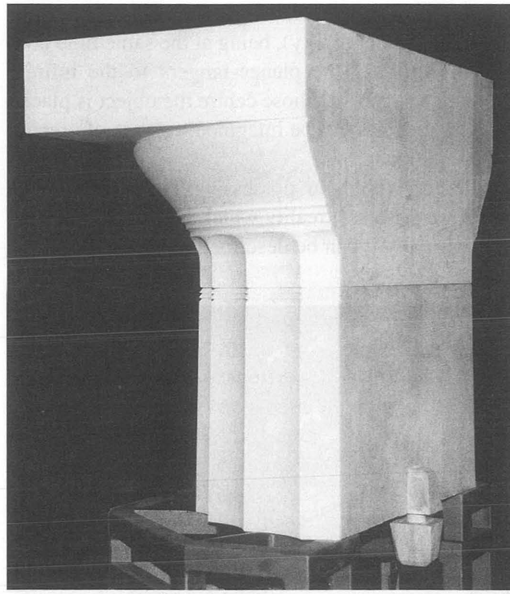

Figure 13
Politecnico di Bari-CMS (Zogno, BG): General view of the 9A capital replica.

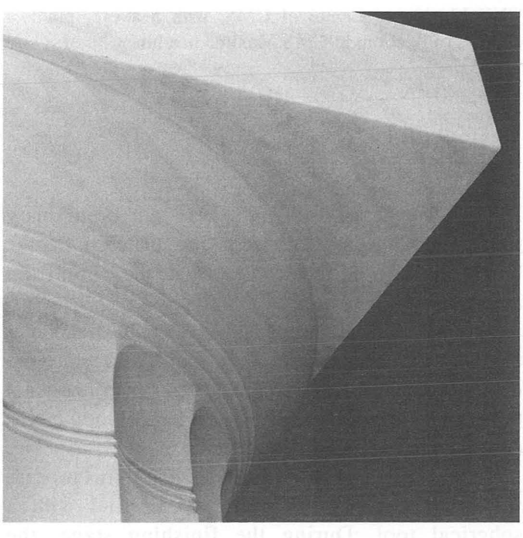

Figure 12
Politecnico di Bari-CMS (Zogno, BG): Detail of the 9A capital replica's echinus and hypotrachelion.

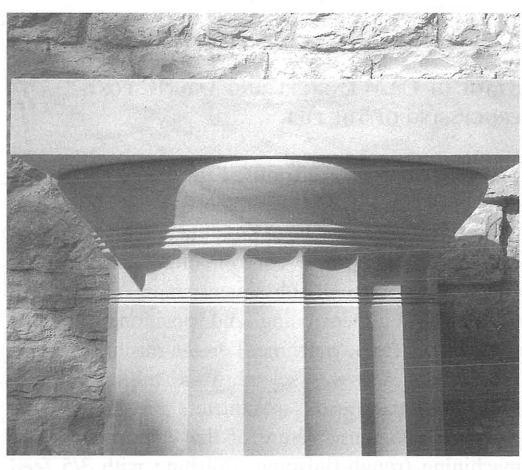

Figure 14
Politecnico di Bari-CMS (Zogno, BG): General view of the 9A capital replica.

Notes

1. The research project was co-financed by MURST and the Bari Polytechnic Institute, and by CMS SpA Costruzione Macchine Speciali, Zogno (Bergamo)
   Scientific Directors
   — prof. Claudio D'Amato Guerrieri, Bari Polytechnic Institute
   — prof. Giorgio Rocco, University of Chieti
   Survey
   — arch. Safoklis Alevridis, Commission for restoration of the temple of Apollo at Bassae
   Executive project (CAD processing)
   — Vito Cascione, Bari Polytechnic Institute
   — Giuseppe Fallacara, Bari Polytechnic Institute
   Executive project (CAM processing)
   — Oliviero Ghisalberti, CMS
   Technical Director
   — Gian Paolo Margheriti, CMS
   General coordination
   — Vartelio Migliorini, CMS
   Realization
   — Festa & C., Ghisalba (Bergamo), with 5 feed-drive NC machine Maxima from CMS.
2. The term «replica» in architecture differs substantially from that of pictorial or sculptural replica, which instead always implies execution by the hand of the same artist. In architecture the significance of replica is limited to compliance with the original project, also executed in its first realization by other hands; and the question of authenticity becomes irrelevant at the moment in which the individual element is acknowledged no independent value as work of art.
3. Cfr.W. B. Dinsmoor, «The temple of Apollo at Bassae», Metropolitan Museum Studies IV, New York, 1932–33.
4. In the design stage, analysis must be made of the problem of congruency between the shape of the tool, the characteristics of the machining path and the final shape to be obtained, in order to perform good machining and to avoid the co-penetration of tools and material.
5. Tool repertoire.
   The characteristics of the machining tools vary in relation to three main parameters:
   — type of machining (preliminary rough-machining stage, finish stage);
   — geometrical characteristics of part to be realized;
   — physical characteristics of material to be machined;
   — speed of feed and rotation of the milling cutter.
   Types of tools:
   — milling cutter for shape (contouring and incision),
   — cylindrical milling cutter (machining on cutting edge),
   — spherical milling cutter (machining on side and end of cutting edge).
6. Finish. The type of run selected for the finish is called «scanning».
   In this case the tool proceeds by successive passes, parallel along the entire surface of the piece, with the depth of each progressive deepening calculated automatically.

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The construction techniques and methods for organizing labor used for Bernini's colonnade in St.Peter's, Rome

Maria Grazia D'Amelio

The diary of Pope Alexander VII Chigi (1657–1668) is full of references to the ongoing construction work on St.Peter's Colonnade (1657–1668), and he frequently complained that the work was «moving very slowly» and urged the people in charge to work more quickly.[1] He clearly wanted to see the colonnade finished within the 60 months scheduled for its completion by the Congregazione della Fabbrica di San Pietro, for which Gian Lorenzo Bernini (1598–1680) was paid 60 scudi a month throughout the whole period of construction,[2] Figure 1.

The five years allotted for building this impressive structure (which at the time was compared to ancient Roman buildings) was not over optimistic, particularly if one considers that the work was originally supposed to be done by the Reverenda Fabbrica di San Pietro. The Fabbrica had been in charge of construction and maintenance in St.Peter's since the early 16^{th} century.[3] It was a very efficient organization, with such a highly skilled technical staff that it was able to finish building St.Peter's cupola in just 22 months, quite a technological feat for the late 16^{th} century. Like many other organizations set up throughout Italy to promote religious architecture, during the 17^{th} century the Fabbrica had an increasingly strong influence on Roman construction in general, in terms of artistic choices, management policies, and spreading architectural knowledge. This was also because the Fabbrica's architects, master builders and highly specialized workmen were quite mobile, meaning that they worked on other construction sites around Rome, spreading their technical knowledge in an osmotic fashion. St.Peter's Basilica and the whole area around it were one enormous building site, equipped with avant-garde

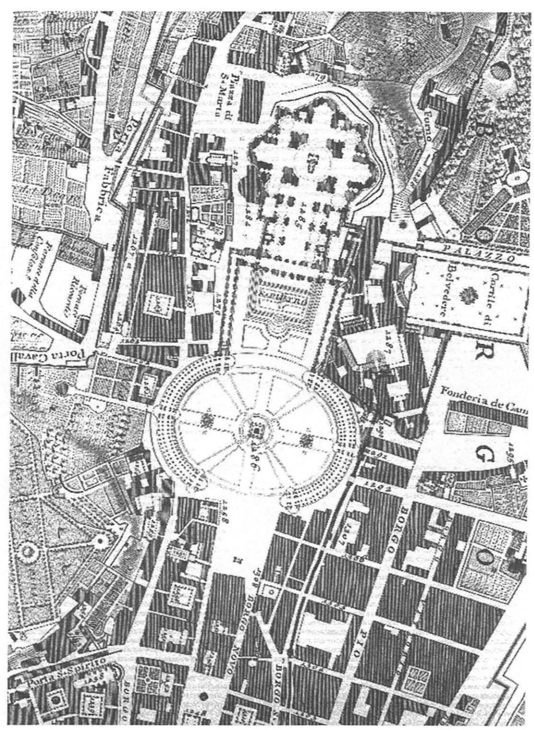

Figure 1
Giovan Battista Nolli. Vatican area. *Pianta di Roma* (Roma 1748), 23, New York 1991: J. H. Aronson Publisher

organization and technology, specialized in designing construction machinery and in the synchronized organization of labor.

The Vatican area also had plenty of resources. There were foundries for making building equipment, stocks of limestone; warehouses full of building materials and equipment, and ample space for working. The Vatican even had exclusive ownership of a river, the Aniene, including its riverbanks and the old Traspontina port, for shipping travertine to Rome from the quarries in Tivoli.

In spite of this supportive framework and the careful scheduling of work phases set up right after Bernini was given the job (July 31, 1656), it took twice as many years to complete this «great . . . Theatre» than had been planned.

It was a mistaken judgement, one of the few dark moments of the enterprise, and was the result of some unwise decisions made when work began.

Alexander VII was counting on this building project to give new life to Rome's economy, which had become dramatically stagnant after pestilence broke out in June 1656. He therefore tenaciously opposed criticism to the project, which some considered unsuitably «grand and ornate» and repeatedly urged the people in charge of the work to procure everything that was needed for the building site,[4] Figure 2.

To speed things up, the Pope also dealt personally with administrative issues connected with the project (giving audience to the *magister stradorum* Domenico Jacovacci, for example) and with its planning, organizational and economical aspects, conferring repeatedly with Bernini, with Mons.Luca Holstenio, Don Flavio Chigi and Father Virgilio Spada.[5]

His impatience was triggered by the knowledge that construction work began substantially after the decision to build the colonnade, a time lag clearly recorded in the memorandum drawn up (probably in August, 1656) by the Fabbrica foreman, Pietro Paolo Drei.[6] According to Drei, it took fully three months to

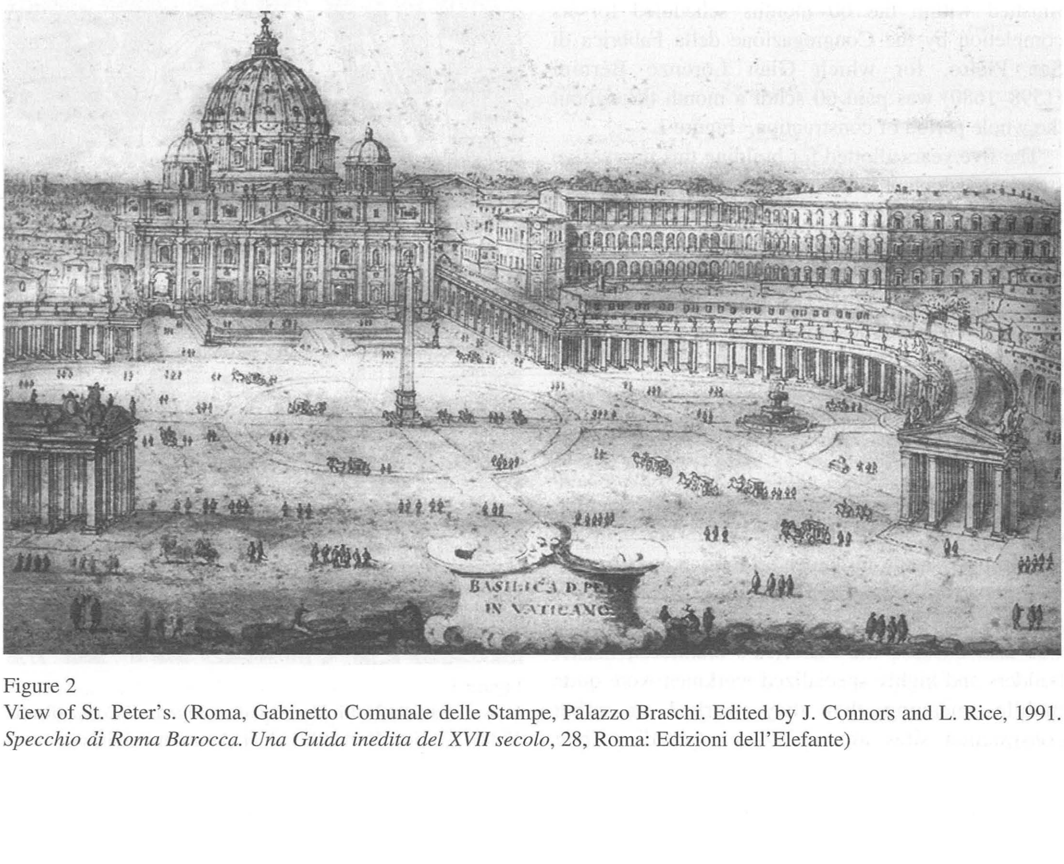

Figure 2
View of St. Peter's. (Roma, Gabinetto Comunale delle Stampe, Palazzo Braschi. Edited by J. Connors and L. Rice, 1991. *Specchio di Roma Barocca. Una Guida inedita del XVII secolo*, 28, Roma: Edizioni dell'Elefante)

lay in stores of tools and construction equipment. He clearly favored the initial decision to start working in an area where no demolition would be necessary, the northern section of the colonnade, towards Porta Angelica. He also suggested that, when calculating the work timing, it would be wise to bear in mind how long it would take to clear out the houses which would have to be torn down to make space for the southern section of the colonnade. These houses were all in the Stelletta and Arcipretato areas and expropriating them had cost just over 50.000 scudi, a very large sum. (By comparison, the Fountain of the Four Rivers in Piazza Navona cost «only» 30.000 scudi). The Fabbrica managed to recover some of the money by selling materials left over from the demolished buildings, either as partial payment to the master masons or else to third parties. This is what happened with the tufa stone taken from Raphael's famous palace (which was knocked down in April, 1671) and the wood from Ferrabosco's tower (demolished in 1660).[7]

Pietro Paolo Drei (an architect who had considerable experience in supervising building projects, since he had worked on the Fountain of the Four Rivers and Sant'Agnese in Agone) assumed that the construction site would be completely controlled by the Fabbrica, using exclusively Fabbrica workmen.

He therefore suggested that buying the freestone from the merchants, and compelling them by contract to provide a specific quantity of material each year, would give the best results. The quality of the travertine would also be specified in the contract. The blocks of stone would be transported from the quarries in a roughly cut state and then be worked on the construction site by crews of Fabbrica stone masons who, since work for them was scarce at that moment, were «idle» and «looking around for something to do». The finished pieces would then be lifted and put in place by the Fabbrica workmen, known as «sampietrini».[8]

Alexander VII shared this view of how the work should be structured. He was initially against the contracting system, because he believed that «work done cheaply is always less well done».[9] Bernini was also convinced that workmen paid on a daily basis produced better quality work and actually defended this theory in 1665, when discussing work management for his never realized Louvre project. The project had to be «well built, otherwise his design would not be successful».[10] On that occasion Louis XIV's minister, Jean Baptiste Colbert, remarked that this method produced an uncertain output, so that construction could not follow a schedule «like it could with contract work», when workmen are not paid on a time basis but according to production.

But, returning to the colonnade, Drei's concluding remark was that all these various aspects of the building site would have to be decided quickly, because «we must make the most of favorable seasons, and particularly of next autumn, because once it is over, the beginning of the year will be difficult and we will have lost an opportunity for employing many poor people».

It was a prophetic remark, since work began only a year later. This long and complex project had just begun, an enterprise, which was to transform the square (considered «outside of every rule in architecture», into the Church's tangible embrace of the faithful,[11] Figure 3.

lthough the project itself had still not been completely defined at that point (the final details were decided late in 1657), on May 28, 1657 the architect Marcantonio De Rossi notified the Fabbrica treasurer

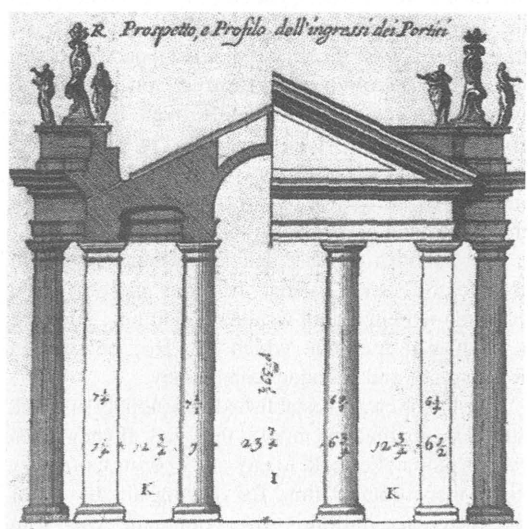

Figure 3
Carlo Fontana, 1694. Section of the Colonnato di San Pietro. In *Templum vaticanum et ipsius origo*, 105, Roma: ex typographia Jo. Francisci Buagni

that «the first boatload of travertine» had arrived».[12] The large ilex rulers used to measure the plan were paid for on July 13;[13] while on July 23 Cardinal Barberini was asked to buy the equipment and materials needed to «start work on the portico» and Bernini was asked to increase the number of workmen (known as *scoccioni*) employed in the travertine quarries.[14]

By that date the system for organizing labor on the building site had already been outlined in the «*Nota delle provvisioni che si devono fare per li Portici*»,[15] which reutilizes a method already used successfully on the Fountain of the Four Rivers a few years previously.[16] This fountain was built in the years right before the 1650 Jubilee, a period when construction was going on throughout Rome at a hectic rate. The churches of St.John Lateran, Santi Luca e Martina, Sant'Ivo alla Sapienza and the Cornaro Chapel in Santa Maria della Vittoria were all being worked on in that period, along with Palazzo Pamphilj, the Basilica of St.Peter's and other, more ordinary buildings.

The *Nota* provides guidelines for making sure that provisions of construction materials would be continuous, a crucial issue if the work schedule was to be respected. To save time, the Fabbrica representatives ordered that employees should go to the Tivoli quarries as soon as possible to choose the travertine blocks, then have them shipped to Rome, where they would be measured and cataloged. Finally, travertine blocks left over from other construction work, which were stored in Piazza Santa Marta (behind the Basilica), were to be moved out into the square, so that workmen could start finishing them, Figure 4.

The Fabbrica likewise decided to build shelters in front of St.Peter's so that the stone masons could continue working in all weather conditions. This was a mediaeval tradition, which can also be seen in Renaissance and Baroque iconography.

The Fabbrica also established a monopoly on brick supplies, so that the bricks that had already been made could not be sold to any other construction site, since the estimated time for making all the bricks necessary for building the colonnade was four months. In any case, the brick kilns (which were all concentrated in the area behind Porta Cavallegeri, near St.Peter's) were mostly active in the summer months, when the heat speeded up the bricks' drying

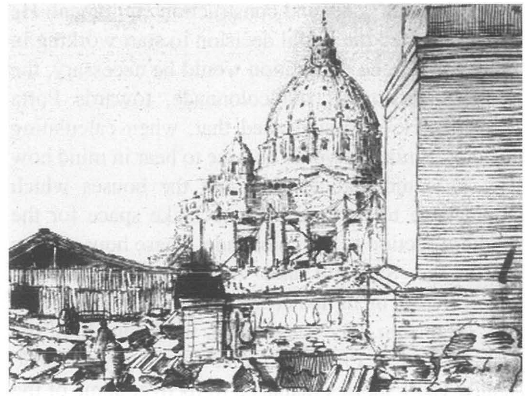

Figure 4
View of St. Peter's. (Wolfenbüttel, Herzog August-Bibliothek, cod. Guelf. 136, Extrav., fo 27. Edited by H. Hibbard, 1971. *Carlo Maderno and Roman Architecture 1580–1630*, 53 b, London: A. Zwemmer Ltd)

process, before they were fired. Therefore, they could only start production in the spring of 1657, which further delayed the start of work on the colonnade. This had already happened in 1656.

A monopoly was also established for the tufa blocks (used for the walls and foundations), for the *pozzolana* and for all, and I stress all, of the lime produced in Rome and Tivoli during that space of time.

Incidentally, the lime was delivered as quicklime in clumps, and then slaked in a big pit dug in the square, near the façade, where it was «more convenient for the work». The method used to slake the lime involved transporting enormous amounts of water into that area of the construction site. For example, it took 1700 liters of water to hydrate 500 kilograms of quicklime, and this would produce just one cubic meter of lime putty.[17] Apart from this notable amount of water, the building site also needed water to wet materials before using them, to mix mortar, plaster, pigments and sizing, to cut and polish stones and to grind down floors. The water was provided by a special connection to the Acqua Paola aqueduct.

The *Nota* ends with an order to procure the wooden planks (known as «piane») used for the floor systems of the provisional structures, and also to procure all of the implements needed by the workmen.[18] The wood and implements were unusually easy to obtain,

because in 1657 very few construction sites were open in Rome.[19]

An estimate of costs for contracting the travertine quarries was also drawn up in 1657. This «Nota della spesa» calculates what it would cost to hire a crew of men to quarry one thousand cartloads of travertine (about 335 cubic meters of stone).[20] The estimate for six months' work was around 1.100 scudi. This included all of the equipment (hoes, pickaxes, poles and sledgehammers of every size, ropes and pulleys, different cuts of wood for provisional structures, and a pump for removing water from the pits) and the wear and tear it would be subjected to, plus animals for transport, renting the quarry, trips to Rome and back to bring the necessary instruments, and living expenses. The estimate also points out that the cost could be reduced if the contractor decided to manage more than one crew of quarrymen, because in that case a lot of the equipment could be shared.

The best quality of travertine came from the Tivoli quarries, but merchants also delivered travertine blocks from Fiano and Monterotondo to St.Peter's. These were considered of inferior quality and Bernini regularly protested about them in the payments,[21] Figure 5.

Depending on the season, the travertine blocks were either brought to Rome on carts drawn by a team of buffaloes or shipped on boats pulled by a team of oxen, who plodded along the riverbanks of the Aniene and the Tiber. To facilitate the oxen's' passage, the Fabbrica had the banks of both rivers cleared of trees all the way up to the Traspontina port. Prisoners were sometimes used to do this kind of maintenance (which was essential for the swift delivery of travertine supplies), serving part of their sentence with the work.

An enormous quantity of freestone was needed for the project, considering that it was used for the whole architectural order of the colonnade (base, column and entablature) «as well as the bases for the statues» and for the statues themselves, according to the life-size model which Bernini had built on the square.[22]

Bernini regularly used these models for his work, «because objects don't appear just as they are, but also in proportion with the objects next to them, and this relationship changes their appearance».[23] He was openly imitating Michelangelo's approach for the coping on Palazzo Farnese. Michelangelo «made a model, and then put it in place height-wise, where it looked so small that he decided to almost double it in size, producing the superb aphorism that distance is an enemy, against which one must fight in an open field».[24]

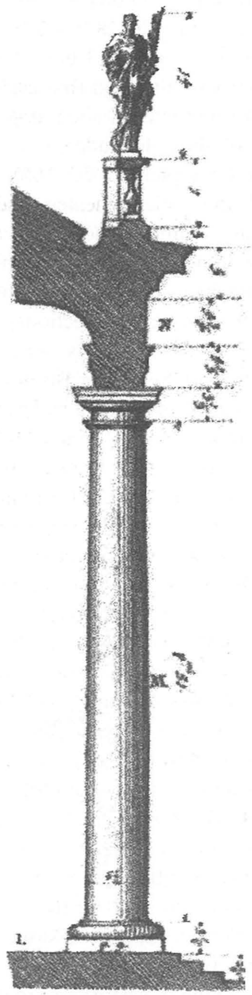

Figure 5
Carlo Fontana, 1694. The *architectural order of the Colonnato di San Pietro. In Templum* vaticanum et ipsius origo, 105, Roma: ex typographia Jo. Francisci Buagni

In spite of the virtually synchronized way that the various phases of construction had been organized, a kind of impatience with the slowness of the work appears in the documents from the very beginning.

Alexander VII's diary entries start to have an imperious tone: «March 28 (1659), Friday, this evening we have ordered Luigi Bernini (Gian Lorenzo Bernini's brother and first assistant) to start work on the Borgia tower as soon as possible and to speed up work on the colonnade».[25]

With reason: on September 20, 1658, for example, «24 columns of the Vatican theater» had been raised. This meant that in slightly more than a fifth of the time allotted to build the colonnade, only one eighth of the Porta Angelica side had been completed (since the work was done in circular sections).[26]

One of the system's flaws was providing a continuous supply of freestone. Because of this lack of material, two years after work had begun there was a surplus of stone masons («men and boys») and some of them risked being fired by the Fabbrica.[27] The travertine was not delivered for various reasons: bad weather conditions in the spring of 1659, but also because contracts with the merchants had not been renewed. Therefore only small quantities of the stone arrived in Rome and were frequently sold to other building sites, such as Sant'Ivo alla Sapienza.[28] As a result of this, workmen would spontaneously abandon the colonnade building site, looking for more permanent employment. In September 1659 as many as 20 workmen disappeared, and Alexander VII ordered that «the missing ones should immediately be sought out with the greatest care and sent to jail».[29]

To overcome this stalemate, a new system was developed, whereby the crews of Fabbrica stone masons only worked on part of the travertine, while as of October 1658 some of the stone masons were given job contracts.[30]

Another part of the travertine was to be worked and finished by the merchants themselves, who therefore had every interest in quarrying firmer, more compact blocks (to avoid having to pay for the expensive patching up of cracks in the stone), which were closer in size to their final shape (to save on transport).[31]

The first contract was assigned to Andrea Appiani. The Congregazione della Fabbrica di San Pietro had decided this during a meeting held on May 25, 1659. Appiani was to make eighteen or twenty columns «of the second row (. . .) with all of the materials and work». The technical specifications listed in the contract are very detailed: the columns must be in Monterotondo travertine, the shaft must be composed of sections made out of a single block of stone, alternated by sections made up of two pieces. No more than two pieces could be used for each of the plinths, bases and capitals.[32] The Fabbrica, on the other hand, committed itself to renting a covered working space on the square to Appiani, moving the rough stones and to transporting and installing the finished blocks.[33]

The quantity of columns in the contract is oddly variable. Appiani had to produce either eighteen or twenty columns, and the uncertainty of this figure was related to the daily work system used by the Fabbrica stone masons. As mentioned, this system makes it is impossible for the work to follow a strict schedule. The contract therefore foresees the possibility that if less than twenty columns «with a diameter of six *palmi*, eight *oncie* and one fifth» (about 135 cm) should be needed for the second row, Appiani would be obliged to build some columns for the second row. In that case, since the columns of the third row have a bigger diameter, the work itself and the rustic stone would be paid more.

Other contracts were to follow. Appiani signed one (Nov.25, 1659) to dig the foundations. Bonifazio Perti signed a contract (Dec.16, 1659) for part of the entablature and Carlo Pervisani (Dec.22, 1659) for some pieces of the architrave,[34] Figure 6.

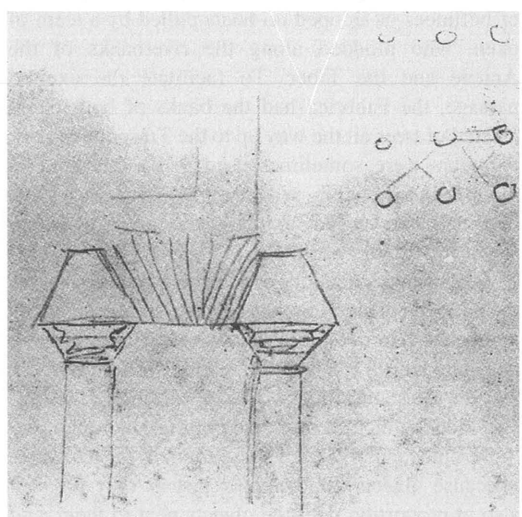

Figure 6
Detail constructive of the Colonnato di San Pietro. (Roma, BV, cod. Chigi a 119. Edited by F. Borsi, 1980. *Bernini architetto*, 73, Milano: Electa)

Work seemed to progress more rapidly in this new structure. For example, the stone masons and bricklayers were very well synchronized, as can be seen in Benedetto Drei's *Diario dei lavori dei Portici circolari*, written between September 4, 1659 and December 13, 1662.[35]

Just to get an idea of what went on work-wise every day, on September 17, 1659 the workmen were used to

— Position the last four segments of the shaft of the second column on the second row of the southern side of the colonnade towards Borgo;
— move the scaffolding, consisting in wooden structures composed of vertical elements (called *candele*), with many joists on different levels and mounted on wheels, so they could be moved wherever needed (see table X in *I castelli e ponti di Mastro Nicola Zabaglia,* Roma 1743);
— transport one of the many *antenne* (a device used for raising weights, similar to a modern crane) used on the building site beside the base of the third column in the third row in the middle towards Borgo: It was to be used to lift some column segments which the stone masons had just finished. Moving the columns with the *antenna* was easy because it was not fixed in the ground but mounted on a thick board with logs underneath, which functioned like wheels (see table VII of Zabaglia 1743; the author sketches and describes the *antenne* used for the colonnade);
— install on the base mentioned above four segments of the shaft, reaching the height of 13 palmi (about 2.90 meters; each segment weighs on average about 2 tons);
— position the capitals of the two pilasters of the second row in the middle towards the Vatican Palace;
— close the brick arch above the architrave of the entrance in axis with the obelisk;
— position the base of the last pilaster of the third row from the Vatican Palace to the center of the colonnade;
— install the plinth of the fourth column of the second row from the Vatican Palace to the center of the colonnade;

And on the following day some of the many operations carried out included:

— finishing the shaft of the second column of the second row of the northern side, towards Borgo, positioning 9 missing sections;
— installing a piece of cornice above the entrance in the northern wing and the capitals on two pilasters;
— positioning the plinth and base of the fourth column of the second row, from the Vatican Palace towards the middle of the colonnade;
— installing two sections of the third column of the second row, from the Vatican Palace towards the middle of the colonnade;
— assembling and bracing the *antenna*, which was about sixteen meters high and made of wooden beams connected by vertical clamps, with a transversal section called the *falcone*.[36] The *antenna* was connected to a pulley with a vertical spindle used to wind up the rope connected to the object that had to be lifted.

Work therefore continued at an impressive rate, and reading the diary one is amazed at how easily they shifted about the great construction machines and mobile scaffolding, which were far more versatile to use than fixed scaffolding, Figure 7. When it rained, and the bricklayers were therefore unable to work, they were sent to the stone masons' shelters, where they picked up marble chips left over from working the travertine, which were then sent to the limekilns to be melted down. In spite of all this activity, between late 1660 and early 1661(before the southern side of the colonnade was started) there was still some talk of how to speed up the work pace.

Father Virgilio Spada in particular suggested that the new wing's foundations should be dug immediately, so that the finished columns could be installed,«otherwise it would be hard to distinguish between the mass of finished and unfinished things in the square».[37] Alexander VII had quite a different opinion (so did Bernini) and wanted the Porta Angelica side of the colonnade to be completed before starting work on the other side. But Spada's view was supported by the fact that it would take a long time to finish the Porta Angelica wing, «since it will be necessary (. . .) to secure the walls so that they don't damage the stability of the palace nearby,

Figure 7
Lievin Cruyl, 1666. View of St. Peter's. *Prospectus Locorum Urbis Romae Insign(ium)*, Roma: Romae Typis Joannis Baptistae de Rubeis (Edited by J. Connors and L. Rice, 1991. *Specchio di Roma Barocca. Una Guida inedita del XVII secolo*, 17, Roma: Edizioni dell'Elefante)

so we must consider that making this wing is like a slow fever and we must be patient, medicating it bit by bit».

During that period Spada was also busy contracting the masonry work, trying to save forty to fifty thousand scudi on the original estimate for the southern side of the colonnade.[38]

This proved to be quite difficult, because the master builders had strong financial reasons for not wanting to build the colonnade's foundations and roofing. These were, in fact, the least profitable parts to build of any structure, and the hard work and poor pay they required were usually balanced by the more remunerative solid parts of the building, which of course don't exist in a colonnade

A contract was finally signed early in 1661 with four master masons, Simone Brogi, Gio Albino Augustoni, Giacomo Pelle and Piero Ostini, who agreed to finish the first side of the colonnade and to build the new one, providing «all the materials».[39]

In this kind of agreement, the master builders are the legal contractors of the work, making a commitment both to do the work and to procure all the materials and equipment needed for the construction,[40] Figure 8.

These master builders then rented the materials and equipment from the Reverenda Fabbrica di San Pietro. Many construction sites in Rome during the Renaissance and Baroque period did this, because small crews of workmen could not afford to buy and maintain scaffolding and machinery, ropes, hardware and cranes. The Fabbrica, as a result of the great

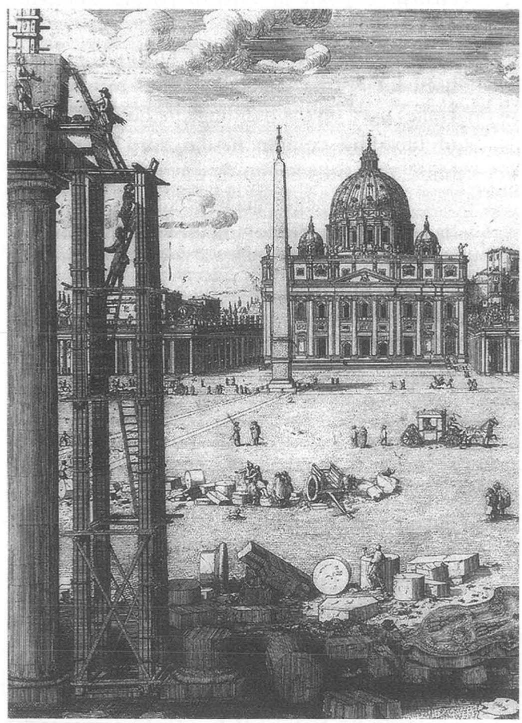

Figure 8
Lievin Cruyl, 1666. View of St. Peter's. In *Prospectus Locorum Urbis Romae Insign(ium),* Roma: Romae Typis Joannis Baptistae de Rubeis (Edited by J. Connors and L. Rice, 1991. *Specchio di Roma Barocca. Una Guida inedita del XVII secolo*, 18, Roma: Edizioni dell'Elefante)

building activity in St. Peter's during the 16th century, owned a huge quantity of equipment, machinery and building materials, which were kept either in the numerous storerooms inside the Basilica or just nearby.

Equipment consigned to the workmen was recorded by the Fabbrica steward in the *Libro delle Robe Prestate* a list of which included details such as quantity, what state the equipment was in and its estimated value. This equipment was not really «borrowed»: the book actually lists the Fabbrica's sale of building materials (lime, *pozzolana*, wood and freestone) and renting tools and machinery (scaffolding, pulleys, hoists and *antenne*). When these objects were returned, another estimate of their value was made and written down in the book, noting wear and tear caused by use. To the cost of this wear and tear was added the full cost of equipment which could no longer be used. The master masons paid for renting the equipment and for buying materials, and the Fabbrica steward sent a note with the sum they owed to the Fabbrica accountant, which was then detracted from their earnings.[41]

On March 28 of that year (1661), Drei recorded in his diary that «We have stopped directly employing the Fabbrica stone masons and started contracting the work».[42] On this date, therefore, the Fabbrica completely changed their work system, to resolve problems which they could not resolve by managing the workmen directly.[43] With the old system, the Congregazione della Fabbrica had to shoulder costs which included paying for eight crews of stone masons, each composed of six men, for the bricklayers, for horses to pull the carts and for the hoists, for the wear and tear of equipment and building machinery, for transporting the stone. And these costs in particular had become unbearably heavy, especially in proportion to how slowly the work progressed during the period when the «Fabbrica did everything by itself».

The Fabbrica was admitting to an operational crisis which, though it did not greatly affect the works final quality, clearly revealed that, at least in terms of time and costs, the initial planning was a complete failure.

Notes

1. Alexander VII's diary (Diary hereafter) is in the Vatican Apostolic Library (BV), cod. Chigi O IV 58, f. 19, col. 2; for the sections quoted here, see Morello 1991, 321–340.
2. BV, cod. Chigi H II 22, f. 126. Bernini did not initially receive any extra payment for the work (except for the «molding on an arch, which will be used for the whole portico»), because it was supposed to be part of his duties as Architect of the Fabbrica di San Pietro, a position he held from 1629. The Fabbrica's foreman and steward supervised the construction site (BV, cod. Chigi H II 22, f. 136–137).
3. According to Moroni Romano 1842, 253, «fabbrica» in ecclesiastical terms means «the revenue used for a church's upkeep, covering repairs and ornamentation as well as everything needed for religious ceremonies». For the Reverenda Fabbrica di San Pietro, which in the 16th century was still called the Collegio Fabricae Basilica, see Del Re 1969; Basso 1987; L. Rice 1997, 7–11; Jones 2000, 399–407 and F. Quinterio 1983, 361–378.
4. For opposition to the portico see BV, cod. Chigi H II 22, f. 97; for the epidemic, Gigli 1994, 2: 763; for the need to built a colonnade, see Diary, August 27, f. 21v, col. 2.
5. For how the building site was organized, see BV, cod. Chigi H II 22, f. 102–103.
6. BV, cod. Chigi H II 22, f. 102–103. The document is dated by Pesco 1988, 45. Drei was the Fabbrica's steward from October 1, 1637 to October 30, 1638. He became theFabbrica foreman on November 27 of that same year and kept the position until the day of his death on November 8, 1656, Archivio della Reverenda Fabbrica di San Pietro (AFSP), armadio 26, ripiano D, volume 272 and armadio 26, ripiano E, volume 305.
7. AFSP, armadio 12, ripiano A, volume 63. Raphael's palace at that time belonged to the Priorato dell'Ordine di Malta (AFSP, armadio 1, ripiano B, volume 20). For the decision to sell salvaged materials: AFSP, armadio 11, ripiano G, volume 48.

Usually all the materials salvaged from demolished buildings were either recycled or sold on request to other construction sites. The travertine dismantled from Bernini's bell-tower, for example, was sold to the building sites of the Campidoglio and Sant'Agnese in Agone (AFSP, armadio 26, ripiano E, vol 303).

8. Payments for the work are listed in the stone masons alphabetical account book from May 18, 1657 to October 31, 1659 (AFSP, armadio 16, ripiano A, volume 164).
9. BV, cod. Chigi H II 22, f. 351.
10. Quoted from Chantelou (1665) 1946, 89.
11. For how the project was developed, see Brauer and Wittkower 1931, 64–102; Thoenes (1962) 1998, 11–47; Haus 1970, 7–16; Kitao 1974; Birindelli 1980; Borsi 1980, 64–96; Fagiolo 1982, 117–132; Rietbergen 1982,

295–358; Haus 1983, 291–315; Krautheimer 1987, 71–80 e 175–180; Marder 1997, 82–105; Marder 1998, 126–150; Roca De Amicis 2000, 283–306. For the chronology and for Mons. Luca Holstenio's role, see Pesco 1988.
12. On September 15 Leonardo Agostini wrote in a letter to Cardinal Leopoldo de' Medici: «Although the foundations are being laid with the greatest speed, the final project has still not been decided» Florence State Archives, carteggio artisti 17, f.35. AFSP, armadio 55, ripiano G, volume 336/337.
13. AFSP, armadio 25, ripiano A, volume 31.
14. Diary, f. 50v, col. 2.
15. AFSP, armadio 7, ripiano F, volume 466.
16. Accademia Nazionale dei Lincei and Corsiniana Library, Codice Corsiniano 167.
17. *Manuale dell'Architetto* 1962, 41.
18. A *piana* is a specific cut of timber (between 270 and 335 cm long., 10 and 20 cm wide and 6 cm thick.). *Ischio* is a variety of oak, Scavizzi 1983, 37–42.
19. Pietro Paolo Drei writes that this type of equipment «can be found easily, particularly right now, when many master builders are selling them, because with the present sacristy of construction sites, the equipment is a useless expense», BV, cod. Chigi H II 22, f. 102–103.
20. AFSP, armadio 7, ripiano F, volume 466. Travertine was also quarried seasonally, from October to April, to prevent the spread of malaria among the quarry workers, Scavizzi 1983, 43.
21. AFSP, armadio 38, ripiano D, volume 40. Contracts for the travertine were stipulated with Andrea Appiani, Pierleone Naldini, Giò Francesco Ghetti, Carlo Pervisani, Bonifazio Perti, Pietro Grassi and Pietro Nerli (AFSP, armadio 7, ripiano F, volume 467). This volume also contains references to Bernini's doubts about the various qualities of travertine used.
22. For the model see AFSP, , armadio 42, ripiano E, volume 1.
23. Chantelou (1665) 1946, 83.
24. BV, cod. Chigi H II 22, f. 107r–109v.
25. Diary, f. 119, col. 2.
26. Rome State Archives, Cartari-Febei, busta 191, f. 13 v.
27. BV, cod. Chigi H II 22, f. 158.
28. For the travertine sold to other construction sites AFSP, Armadio 56, Volume 343.
29. AFSP, armadio 56, ripiano A, volume 343.
30. AFSP, armadio 26, ripiano E, volume 327.
31. BV, cod. Chigi H II 22, f. 159.
32. AFSP, armadio 7, ripiano F, volume 467. The accounts for Appiani's work between 1658 and 1662 are in AFSP, armadio 42, ripiano E, volume 3.
33. AFSP, armadio 16, ripiano A, volume 168.
34. AFSP, armadio 7, ripiano F, volume 467.
35. Benedetto Drei, Pietro Paolo's son, was assistant foreman of the Fabbrica from March 1, 1657 to July 13, 1675. The Diary provides abundant information about the timing of the work, AFSP, armadio 17, ripiano E, volume 29.
36. For a description of the *antenna* see Zabaglia 1743, tav. II; tav. VII.
37. BV, cod. Chigi H II 22, f. 183.
38. The «Scrittura di monsignor Spada per l'apula con i muratori» (BV, cod. Vat. Lat. 7939, f. 311–313) is dated by Pesco 1988, 82. The «Scandaglio di tutta la spesa che va in fare il Portico dalla parte verso Cesis» is in BV, cod. Chigi H II 22, f. 201.
39. AFSP, armadio 16, ripiano A, volume164; the masons started to work regularly on February 28, 1661, AFSP, armadio 27, ripiano A, volume 334; for the contract for the southern wing, see BV, cod. Chigi H II 22.
40. For a typical example of one of the many all-inclusive contracts «a tutta roba» see «Obbligo di Giuseppe Bucimazzi di Rocca di Mezzo per realizzare le fondazioni per i portici de San Pietro profondi 12 palmi con tutti i suoi ferramenti» (AFSP, armadio 7, rip. F, vol 466).
41. The list of «Munizioni (building materials and equipment) restituite alla R.da Fabbrica di San Pietro dalli capomastri muratori consegnati al Giacomo Balsimelli fattore» in 1666 (AFSP, armadio 12, ripiano E, volume 6) records the rented equipment and provides measurements of the scaffolding and *antenne* used on the construction site..
42. AFSP, armadio 17, ripiano E, volume 29.
43. «Ragione per qual causa la Reverenda Fabbrica di San Pietro ha dato il restante del lavoro delli portici di scalpellino alli 4 mercanti» («Reasons for which the Reverenda Fabbrica di San Pietro has given the rest of the portico work to the 4 merchants»), AFSP, armadio 12, ripiano D, volume 3.

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